SYMPOSIUM ON CHANNEL STABILIZATION PROBLEMS

Committee on Channel Stabilization

TECHNICAL REPORT NO. 1

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Committee on Channel Stabilization

CORPS OF ENGINEERS, U. S. ARMY

ARMY MRC VICKSBURG MISS

PRESENT MEMBERSHIP OF COMMITTEE
ON CHANNEL STABILIZATION

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* Replacing Mr. R. W. Sauer who resigned subsequent to the second meeting of the Committee.

PREFACE

Establishment of the Committee on Channel Stabilization in April 1962 was confirmed by Engineer Regulation 15-2-1, dated 1 November 1962. As stated in ER 15-2-1, the objectives of the Committee, with respect to channel stabilization, are:

- a. To review and evaluate pertinent information and disseminate the results thereof.
- b. To determine the need for and recommend a program of research; and to have advisory technical review responsibility for research assigned to the Committee.
- c. To determine basic principles and design criteria.
- d. To provide, at the request of field offices, advice on design and operational problems.

At the first and organizational meeting of the Committee, held 25-27 April 1962, a number of papers were presented by Committee members from Corps of Engineers District and Division offices dealing with United States rivers having major channel stabilization problems. These papers described channel stabilization works, pointed out the principal problems encountered in the design and construction of such works on various rivers in the United States, and discussed phases of this work for which additional or improved criteria are needed. In addition to these papers, representatives of the U. S. Army Engineer Waterways Experiment Station discussed research on channel stabilization, including proposals for a research program.

This report, volume 1 of a Symposium on Channel Stabilization Problems, comprises the papers presented at the first meeting of the Committee. In addition, it includes a summary of the studies and investigations suggested by the Committee members. It concludes with a discussion of the use of hydraulic models to study river sedimentation problems.

Volume 2 of the Symposium will contain other papers by Committee members on the same general subjects.

Copies of this and future reports of the Committee on Channel Stabilization may be obtained from the U. S. Army Engineer Waterways Experiment Station, P. O. Box 631, Vicksburg, Mississippi.

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CHAPTER I

STABILIZATION PROBLEMS, LOWER MISSISSIPPI RIVER

by

R. H. Haas*

Introduction

1. The bed and banks of any stream are unstable in greater or less degree depending upon its geological environment, discharge, and slope. The purpose of stabilization is to so control the flow that a favorable channel shall form in places where it has been inadequate. Re-forming the channel often alters the natural tendencies and creates new problems. Stabilization, therefore, is not wholly a question of the application of known hydraulic formula nor too much confidence that the effect will be favorable in all essential details if only theoretical values are attained. Neither is it entirely a proposition of adopting a design or plan that has proved successful elsewhere. Stabilization or regulation of rivers must blend theory and practice to an extent that may yet be imperfectly understood.

2. The Mississippi River is a highly sinuous stream. Below Cairo its length is about twice that of the alluvial valley through which it flows. While geologists have satisfactorily reconstructed the various phases of valley filling, our concern is to evaluate the relation of the various events on the discharge, the valley slope, and the character of the alluvium. This information, together with an understanding of the forces engendered, is necessary for proper planning and construction of stabilization works.

Discharge and stream characteristics

3. The Middle Mississippi has carried the combined flow of the Upper Mississippi and Missouri Rivers since well before the last ice age. The Lower Mississippi has carried the combined flow of the Ohio and Mississippi Rivers since the Mississippi was diverted through Thebes Gap when the last ice age receded some 6000 to 8000 years ago. After this junction the

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Lower Mississippi River formed larger meander loops than those formed by either the Mississippi or Ohio Rivers alone. Some indication that the size of the meander loops bears a direct relation to the valley slope and discharge may be illustrated by the regimen of the White River. The White River, with a small discharge flowing on a slope comparable to that of the Mississippi River, and in similar alluvium, has much smaller meanders. Also ancient traces of the Ohio and Mississippi Rivers before their junction at Cairo developed much smaller meanders than could be tolerated by the combined discharge.

4. The contribution of additional volumns of water by major tributaries also affects the discharge-slope relation. The principal effect on the Mississippi River, however, is a slight flattening of the slope above the mouths of the tributaries. The flattening, in some instances, results from the introduction of sediment although, in general, the locally flattened slope is related to the backwater effect. At St. Louis the minimum discharge is about 48,000 cfs, and the maximum about 500,000 cfs. At Hickman, Kentucky, the minimum discharge recorded is about 70,000 cfs, and the maximum about 2,015,000 cfs. At Natchez the minimum discharge is 99,900 cfs, and the maximum 2,046,000 cfs. The discharge of the Lower Mississippi River is about four times that of the Middle Mississippi. The stage differential between extreme high and low water is approximately 48 ft at St. Louis, 60 ft at Cairo and Natchez, and 22 ft at New Orleans.

Slopes

5. The slope of the Middle Mississippi River between St. Louis and Cairo averages 0.56 ft per mile at low water. The present overall slope in the 966 miles of the Lower Mississippi River from Cairo to the Gulf is the least it has attained in its recent geological history. The total fall at bank-full stage is 316 ft or 0.33 ft per mile. At low water the fall is about 280 ft or 0.29 ft per mile. The valley slope between Cairo and New Madrid (Sikeston Ridge) is about 0.2 ft per mile; below New Madrid to Helena, Arkansas, the average low-water slope is 0.5 ft per mile. From Helena to Natchez the average slope is about 0.4 ft per mile, and in the southern section, below Natchez, the average low-water slope is only about 0.05 ft per mile. Within these general divisions, local slopes are subject to sharp changes, especially between bendways and crossings. Where the

river is wide and shallow and slopes are steep, the river tends to develop reaches of divided flow with shifting bars, towheads, and islands.

Alluvium

6. The Mississippi above New Madrid flows through the most ancient deposits in the valley. These deposits are predominantly coarse sands and fine gravel. Below New Madrid the alluvium is less coarse, and the river is actively reworking alluvial fan deposits laid down by ancient braided streams. In the southern part the materials encountered are, for the most part, deep deposits of fine-grained clays and silts.

The stabilization problem

7. As stated before, channel characteristics, and therefore the problems, are closely related to discharge, slope, and the alluvium. Essentially there are four distinct segments which dictate to a greater or lesser extent the nature of stabilization required. For the purpose of detail planning, these segments can and must be subdivided further. In uniform material, the Mississippi River would meander downstream in regular loops. Since, however, the materials encountered are heterogeneous, the meander pattern is altered by local resistant bank materials which alter the stream characteristics and influence the downstream alignment. Irregularities in the migration pattern often result in the formation of a distorted or abnormal meander which eventually leads to a natural cutoff of the loop. Stabilization of a bendway on its natural curvature is not difficult with the articulated concrete mattress. In order to insure continued effectiveness, however, the revetment must be supported by a fixed alignment above and below. For this reason it is generally desirable to complete the stabilization works downstream from a fixed point, directing the flow across the crossings, as far as this is possible, and protecting successive bends in an orderly sequence. Stabilizing a bendway does not in itself, however, insure that the channel will remain in the bendway unless equilibrium of the slope above and below is also attained.

8. By far the most difficult problem is the stabilization and establishment of a deep channel in the long, relatively straight sections or reaches. Reaches may be found downstream from locations where the channel impinges against the valley wall, or where successive cutoffs have occurred and the channel is more or less confined or controlled by the interfacing

cutoff meanders whose arms have been filled with ancient clay deposits. A reach, then, is formed when the zone of migration becomes so narrow that a large meander loop can no longer develop. The channel in the lower part of the northern segment of the valley, more specifically between Richardson Landing, Tennessee, and Helena, Arkansas, is marked by a series of such reaches. There are fewer of these reaches in the central segment, Helena, Arkansas, to Angola, Louisiana. Straight reaches in the central segment have resulted for the most part from artificial cutoffs and bluff impingements. Attempts to stabilize reaches in the Lower Mississippi River have often been disappointing because the low water, midbank, and flood stages seldom follow identical flow patterns.

Plan of stabilization

9. The plan for the Middle Mississippi is basically one of contraction with pile dikes and revetment. The plan is well advanced and there appears to be no special problem in maintaining the authorized 9-ft navigation channel. Pile dikes, of course, remain effective only as long as they can retain accretion. Ice damage is often severe. Replacement with stone fill for durability is gradually being accomplished.

10. The Flood Control Act of 1944 authorized stabilization of the channel between Cairo and Baton Rouge. The Master Plan for this project requires 607 miles of revetment, and 102 miles of dikes and dredging as necessary to realign the channel and fill the dike fields where accretions do not form naturally. The Review Report provides for an additional 100 miles of revetment between Baton Rouge and the Head of the Passes.

a. Revetments. Revetments are the principal structure for stabilization, and 77% of the authorization for the project, Cairo to Baton Rouge, is set up for this feature. At the end of the low-water season FY 1962, 445 miles of operative revetment were in place and this feature was 74% complete. The greatest concentration of revetment is planned for the middle sections of the Lower Mississippi River. It is in this section of the river, roughly between Randolph, Tennessee, and Vicksburg, Mississippi, that most of the cutoffs, both natural and artificial, have occurred. These relatively straight reaches, if left uncontrolled by dikes and dredging, would conceivably require revetment on both sides of the river in order to contain the channel. Revetment alone, however, cannot establish a permanent channel. It is in these reaches that the most difficult task of re-forming and developing a single, deep, low-water channel remains.

b. <u>Dikes.</u> Dikes are needed at locations where it is necessary to prevent divided flow, retard bar movement, and establish the channel in a desired location. The greatest concentration of dikes is proposed for the long, wide reaches where the channel has a history of instability. As of this time, 22.4 miles have been completed or 27.5% of all the dikes proposed. The main difficulties insofar as dikes are concerned are the depth of water, the erratic change in direction of flow, and failure to accumulate fill. The absence of accretion appears to result from the fact that most of the bendways have been stabilized and, therefore, material normally accreting in the dike fields has been eliminated.

c. <u>Dredging.</u> Dredging is necessary for realigning the channel, deepening crossings, and filling dike fields which do not accumulate fill naturally at a reasonable rate. While it is difficult to estimate the ultimate amount of the dredging that will be required to produce the 12-ft channel, approximately 75% of the total estimated has been completed.

<u>Maintenance</u>

11. Contrary to common belief, maintenance of the articulated concrete mattress is not out of line with that of most other structures. The overall average cost of annual maintenance is $15,000 per mile of operative revetment. This, of course, varies by districts as well as within the district boundaries. Overall revetment maintenance cost to date has been reduced to approximately 2% of the initial cost of revetment. It should be noted, however, that annual reinforcements required where the channel deepens excessively amount to approximately 1% of the initial cost of revetment, making the total annual maintenance and reinforcement factor approximately 3%.

<u>Potamology</u>

12. Between 1947 and 1952 the Mississippi River Commission (MRC) undertook a series of Potamology Investigations at the Waterways Experiment Station (WES). These investigations have been reviewed by Mr. J. B. Tiffany in a paper entitled <u>Review of Research on Channel Stabilization of the Mississippi River, 1931-1962,</u> which constitutes the Committee on Channel Stabilization's Technical Report 2.

13. In 1957 General Hardin, then President of the MRC, authorized resumption of Potamology Investigations relating to stabilization problems of the Lower Mississippi River. General Orders No. 9, dated 26 April 1957, established the Potamology Board consisting of one member each from the

Memphis, Vicksburg, and New Orleans Districts, one from the Waterways Experiment Station, and the chairman and secretary from the Mississippi River Commission. The District board members were made responsible for assembly and coordination of data required, the WES member was designated a consultant with responsibilities to coordinate phases of the investigations performed at WES. In general the purpose of resuming investigations was to provide practical answers to the most pressing problems.

14. Practically every improvement in mattress and pavement design and construction methods in recent years has had for its objective the elimination or lessening of one or more of the recognized reasons for failure. Although failures occur from a great many causes, the principal ones are listed below.

a. Upper bank failures may be attributed to:

(1) Removal of material landward of pavement by overbank scour.

(2) Removal of material from the pavement subgrade by surface or underground drainage or wave wash.

(3) Mechanical damage to pavement due to boats, drift, ice, etc.

(4) Connection between bank paving and subaqueous mat.

b. Subaqueous mattress failures may be attributed to:

(1) Removal of material from mattress subgrade by scour. It is known that removal of material through the mattress interstices by current action often occurs.

(2) Removal of material from the riverbed and outstream edge of the mattress, that is, deepening of the bed of the channel and encroachment on the mattress resulting in the undermining of the outboard end.

(3) Movement of part or all of the bank due to instability (shear and flow slides).

(4) Mechanical damage to the mattress during sinking operations.

(5) Loss of pavement or mattress by flanking action of the river currents.

15. Any individual failure of a revetment may be caused by any one of these (and possibly others) or combinations thereof. Some failures, particularly those occurring at high-water stages, are so complex as to leave no visible or survey evidence of the cause. However, every effort

is made to determine the causes of all revetment failures in an effort to improve design and construction practices. By far the most frequent of serious revetment failures are due to progressive deepening of the river channel, scouring of material from under the riverward toe of the mattress, and progressive disarrangement, damage, and ultimate destruction of the outer portion of the mattress. This condition, when continued over several years, may result in bank instability and, consequently, failure of the work.

16. In the past the greatest overall loss of revetment has been caused by flanking. This may occur at either the upstream or downstream end. Flanking is usually more critical if it occurs at the upstream edge since it may cause loss of an entire installation. At the downstream terminus, flanking results from eddy action which works upstream to undermine the bank. Downstream flanking losses usually do not affect more than 100 to 500 ft of the work. When practical, revetments are extended over the entire area between the upstream and downstream terminus of caving at all stages to prevent flanking. As the stabilization program progresses, these losses may be entirely eliminated.

17. Initial investigations undertaken by the new board were:

- a. Improvement of the connection between the upper bank paving and underwater mattress.
- b. Investigation of life expectancy of the asphalt pavement.
- c. Investigation of instruments for studying condition of subaqueous mattress.
- d. Stability of revetted banks in the New Orleans District.
- e. Clearing and grading of underwater slopes in the New Orleans District.
- f. Development of a cone penetrometer to facilitate determining bank stability.
- g. Verification of the empirical method of determining slope stability (started 1953).

The results of these investigations are discussed in the following paragraphs.

18. The connection between the subaqueous mattress and upper-bank paving fails when channel deepening occurs and the channel mattress slides down the steepened slope leaving an unprotected area at the connection.

The problem is more pronounced where monolithic pavements (asphalt) are constructed. The most serious failures occurred in the Vicksburg and New Orleans Districts. The Memphis District has solved the problem by going entirely to riprap for pavements, with additional thickness at the junction. Also, both Memphis and Vicksburg have made use of independently anchored "slipmats" of articulated concrete mattress. By this method, the channel mat could slip 3 to 5 ft without breaching the connection. This method, however, is costly. After the study was under way, the Vicksburg District developed a "mat-pulling" device by which it is possible to pull the channel mattress high up the slope where the connection can be kept under observation and repaired, where necessary, by the minor repair contractor. The study was accordingly concluded.

19. The investigation of life expectancy of asphalt paving was undertaken by both the Memphis and Vicksburg Districts:

a. The Memphis District based its study on maintenance records over a period of 12 years. The study concluded that by the time asphalt pavements were 12 years old, 52 percent had to be replaced. While the rate of deterioration for the first few years was minor, the rate increased significantly in later years. Extending the curve, a life expectancy of about 20 years was indicated. Unit maintenance costs of asphalt were more than three times that of riprap. A significant conclusion of the report is that if a 20-year life for asphalt and a 40-year life for the riprap are assumed, the cost of riprap could be economically supported at 2-1/2 times the initial construction cost of asphalt. The report has not been published, although records are being maintained.

b. Vicksburg estimated the life of asphalt paving on the basis of abrasion. A 25-year life was computed. The results of these tests have been published in WES Miscellaneous Paper No. 4-385, March 1960.

As a result of these two studies, no further asphalt work is being constructed between Cairo and Greenville. The extent to which asphalt will be eliminated in the lower sections of the river will depend upon the contractors' ability to furnish barges. Most of the stone comes from quarries on the Cumberland and Tennessee Rivers, although some rock is being economically transported by rail from Little Rock, Arkansas, to near the mouth of the White River and transferred to barges for destinations as far downstream as Old River, Louisiana.

20. The investigation of instruments to locate buried mattress, undertaken by the Vicksburg and New Orleans Districts, is still under way.

a. The Vicksburg District contracted with Geotechnics and Resources, Inc., for underwater investigations with seismographic equipment. A report has been published. While there is indication that the mattress can be located, there is some doubt as to whether the results are accurate. In any event, the seismographic method requires a great number of assumptions and the mathematics required are considered too complicated for practical application.

b. The New Orleans District has recently completed, with assistance of WES, tests of the sonar "Thumper" and "Pinger" manufactured by Edgerton, Germeshausen & Grier, Inc. A preliminary report indicates that this device has promise. The sand bottom and mattress buried up to 5 or 6 ft are distinguishable on scrolls. Edgerton, Germeshausen & Grier, Inc., is confident that with some modifications of their equipment, mattress can be picked up at greater depths. An interim report has been submitted.

21. Stability of revetted banks in the deep, fine-grained deposits of silt and clay in the New Orleans District is still in the preliminary study phase.

22. Some work has been done at Allendale during the past low-water season to determine adequacy of our present equipment for deep underwater grading and clearing. Report has not been submitted.

23. WES has been conducting tests for the development of a cone penetrometer. The results will be reported by WES.

24. In a letter dated 18 February 1953 to WES, the President, MRC, endorsed a program for verification of the empirical method of computing slope stability. This program is being conducted on an annual basis with data furnished by the three lower districts of the Lower Mississippi Valley Division. The latest report, Potamology Report 12-11, dated December 1961, concludes:

> The modified classification criteria have proved reliable in predicting susceptibility to flow failure, in that locations where flow failures have occurred have been predicted to be unstable. However, many locations predicted to be unstable have not experienced flow failure, and it is possible that either the density of the zone A sand may be such that flow failure will not occur, or that the severity of river attack up to this time has not been sufficient to initiate a flow failure.

25. The following <u>additional investigations</u> are under way:

   a. Surveys are now under way in the Memphis and Vicksburg Districts to obtain slope and flow data for studies of specific reaches. These studies, with proper interpretation and application, should provide information to supplement the studies suggested for future investigations for developing criteria for stabilization of reaches.

   b. Hydraulic investigations are under way to study changes in channel cross sections over a wide range of stages to determine the relation of changing depths to bank stability. The field work by the Vicksburg District has been completed, and the data have been furnished WES for evaluation.

   c. A plan for canvassing technical literature relating to regulation of open rivers as it pertains to our problems is being developed by WES.

26. At each subsequent meeting of the board new problems were presented, discussed, and agreed upon. Among the studies proposed for the near future are:

   a. <u>Curvature.</u> Criteria including radius and length of bendway, and curvature to be sought in establishing proper alignment for stabilization.

   b. <u>Crossings.</u> Criteria relating to length, alignment, width, slope, and variations in stage to meet navigation requirements and minimize maintenance.

   c. <u>Contraction.</u> Criteria relating width, slope, and stage to alignment for various stages to meet objective of bringing the discharge into a single stabilized channel having project depth at low water.

   d. <u>Meander pattern.</u> Criteria for alignment in relatively straight reaches to accommodate low- and high-water flows insofar as possible. If not possible, establishment of a basic alignment to which the channel will return at low water.

   e. <u>Loss of caving bank material due to stabilization.</u> Effect on crossings, pointway channel tendencies, movement of bars, effectiveness of dike systems, and effect on sequence of revetment, dikes, and improvement dredging.

   f. <u>Dike systems.</u> Criteria for height, spacing, type, and alignment related to purpose; for example: criteria for dikes in straight reaches may differ from those designed to direct or contract the channel through a crossing between bends.

Conclusions

27. The success of the plan for stabilization of the Lower

Mississippi River to produce a 12-ft channel depends on the progressive adaptation of plans to local conditions, both as they exist naturally and as they will be modified by the works constructed. The completeness to which the increased depth can be attained will depend particularly upon the thoroughness with which theory and practice can be blended to produce a guiding influence on the low-water flow.

CHAPTER II
BANK STABILIZATION AND CONTRACTION PROBLEMS IN THE SOUTH ATLANTIC DIVISION

by

C. P. Lindner*

1. The South Atlantic Division has a myriad of relatively small streams when compared with the rivers with which other members of the Committee on Channel Stabilization work. The streams of greatest size and importance are shown in fig. 1. The Mobile river system, the largest in the South Atlantic Division, has a drainage area of 44,700 sq miles. Of this, 22,800 sq miles are in the Alabama River Basin and 19,970 sq miles are in the Tombigbee River Basin. The Apalachicola River has a drainage area of 19,170 sq miles, which is the second largest drainage area in the Division. The largest flowing into the Atlantic Ocean is the Yadkin-Pee Dee, which has a drainage area of 16,340 sq miles. The rivers in the Southeast pass through geological provinces, some of which do not exist in the areas represented by the other members of this Committee. Except in Mississippi and Florida, the headwater ends of the streams generally lie in the mountain sections, although a few streams originate southward or eastward of the mountains in the Piedmont which is the next geological province that the rivers traverse on the way to the sea. The largest stream that rises in the Piedmont is the Altamaha in Georgia with a drainage area of 14,350 sq miles.

2. In northern Alabama the mountain section consists of what is known as the Valley and Ridge province trending from the northeast to the southwest. Lookout Mountain is a part of this complex. The mountain section to the eastward is the Blue Ridge, which ends essentially in northern Georgia. The mountain section covers the north portion of the State of Georgia and the western portions of the States of South and North Carolina and Virginia. Some of the highest mountains in the east are located here. The Great Smokies, for example, are in North Carolina, as is Mt. Mitchell

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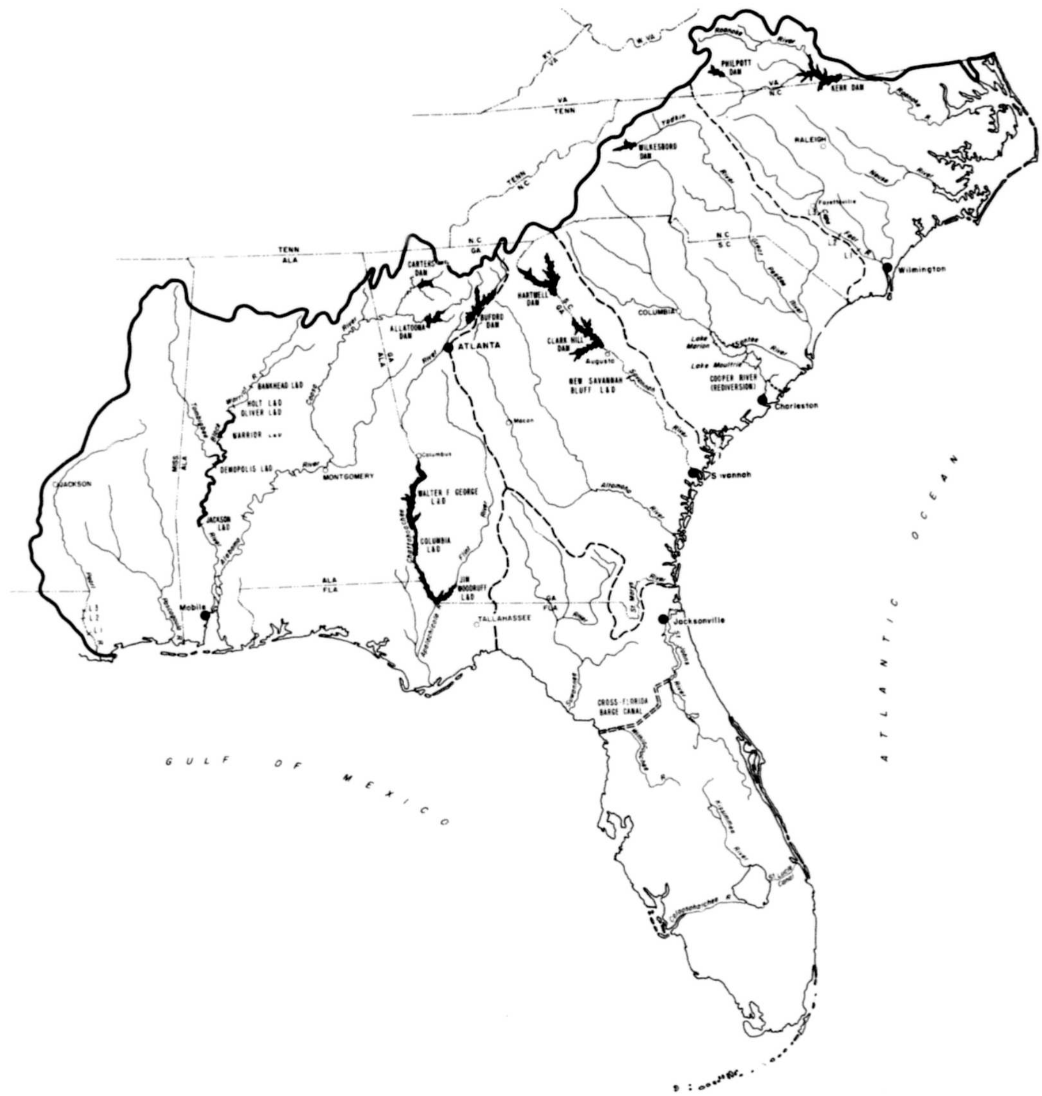


Fig. 1. River systems in the South Atlantic Division

which is the highest peak in the Appalachian system. It is interesting to note that the highest rainfall in the eastern United States occurs in the mountains of Georgia and the Carolinas. At the head of the Savannah River, for example, some stations have recorded average annual precipitations in excess of 80 in.

3. The Piedmont Plateau area extends from the southern and eastern

terminus of the mountain province to a line, known as the fall line, which runs essentially through Demopolis and Montgomery, Ala., Columbus, Macon, and Augusta, Ga., Columbia, S. C., and Fayetteville and Raleigh, N. C. Here the streams plunge over rapids and waterfalls. Seaward of this line is the area known as the Coastal Plain. The Piedmont is covered with a relatively thick blanket of residual soil underlain by weathered rock. Below the weathered rock generally are granite or calcareous rocks such as dolomite, marble, etc. The topography is rolling and hilly with considerable relief. As the slope is comparatively low, the hills are high, and the valleys have substantial width, there are numerous good dam and reservoir sites. The residual soils, however, are quite erodible and, as a result, erosion problems beset the area, especially when streamflows are fluctuated rapidly by peaking operations of power plants.

4. The earthy mantle of the Coastal Plain consists of transported sediments, some of which have become indurated to the extent that they are fairly hard. The surface mantle is generally underlain by limerock that is soft, porous, and cavernous. The region is relatively flat, although in some of the areas, especially Georgia, there is a limited amount of topographic relief in the form of rolling, low hills. There are few good damsites in the Coastal Plain, so that a substantial part of the development of the waterways in this region is by open channel work. In a number of instances, however, locks and dams have been constructed where the topography was favorable and foundations were adequate.

5. As implied above, the streams meander and erode their banks moderately in the Piedmont. Consequently there has been little need to stabilize the banks. But below hydroelectric power plants where the streamflows fluctuate rapidly, the pumping action has caused erosion, and protection at isolated locations has been found necessary. Moreover, where navigation channels have been developed by means of low dams, as on the Tombigbee-Warrior system prior to the present reconstruction program, there is considerable sand movement; consequently, after each flood it is necessary to dredge out the pools and even the lock chambers. Had these low dams remained, it might eventually have proved desirable to provide some bank protection to reduce the shoaling in the waterway. The low dams on the Tombigbee-Warrior waterway are being replaced so that sand shoaling

should be less troublesome in the future. Nevertheless, movement in the stream channel of large amounts of sand is quite common in the Piedmont region.

6. The rivers of the Coastal Plain meander radically except where the banks are composed of indurated material, as along the Chattahoochee River below Columbus, Ga. There has been little call for bank stabilization, however, because most of the land immediately adjacent to the streams is undeveloped and remains wooded. Fig. 2 shows a portion of the Savannah River in the Coastal Plain prior to straightening by cutoffs. Although the meandering shown is rather extreme, it indicates conditions that may be expected to be encountered in this region. The undeveloped nature of the overbank area is rather typical.

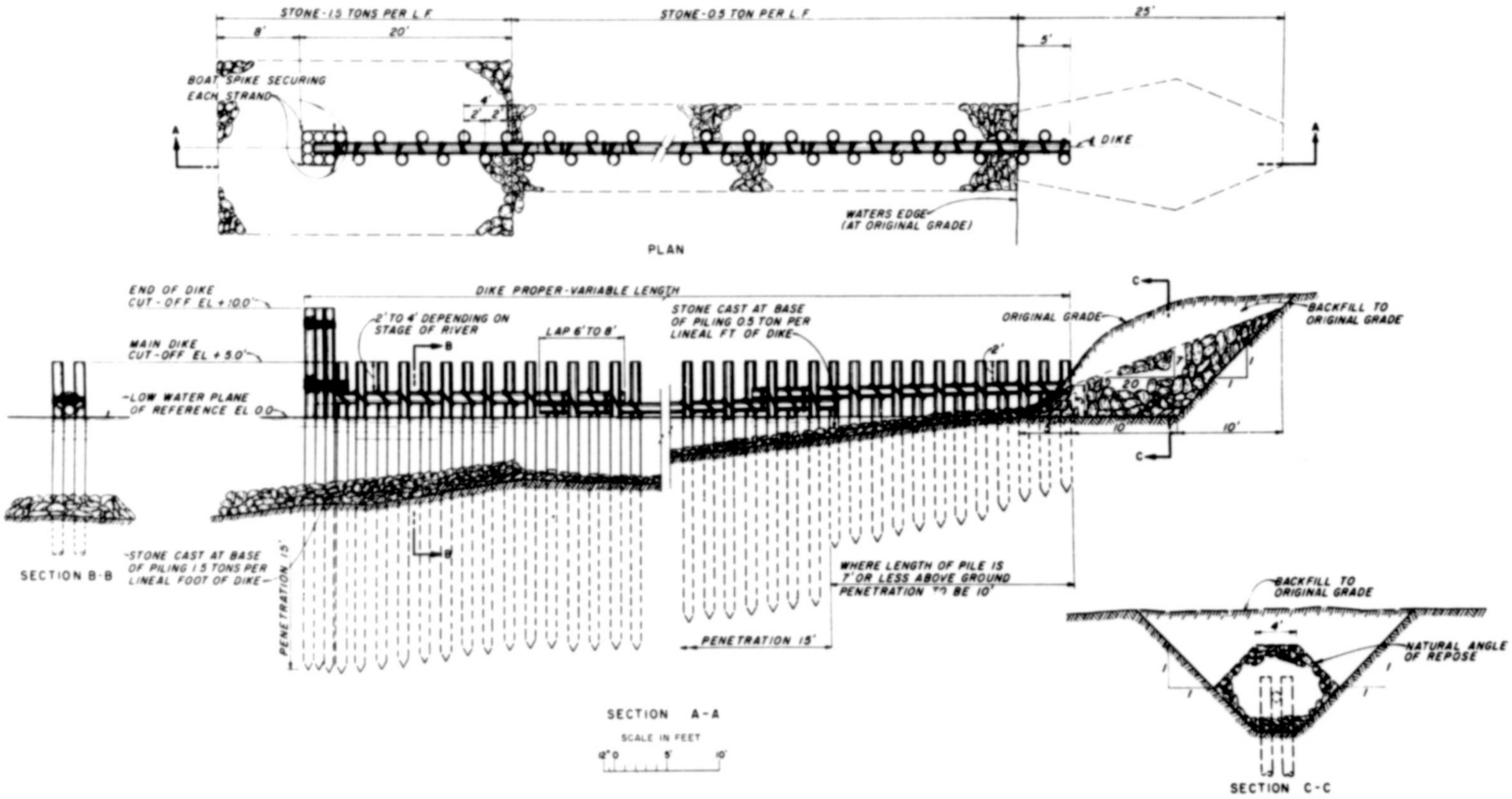
7. The two main problem streams, so far as requirements for channel stabilization are concerned, have been the Apalachicola and the Savannah Rivers. Navigation channels are authorized on these two streams, and the authorized depths and widths are to be provided and maintained by dredging and channel contraction. Thus, the main channel stabilization problem right now in the South Atlantic Division is not bank stabilization as such,

Fig. 2. Example of bends in Savannah River

but the provision of navigable depths and the reduction of maintenance by means of contraction works.

8. On the Savannah River, Hartwell and Clark Hill Reservoirs regulate the flow so that, with the help of the Stevens Creek pool just above Augusta, Ga., to iron out the power waves, the minimum flow at that point will be 5800 cfs. The construction on the open river project is about completed. Dikes and revetments have been installed and cutoffs have been made to produce, direct, and hold the navigable channel. Fig. 3 shows the design features of the dikes and revetments. The Savannah River in this open channel section has an average low-water slope of 0.58 ft per mile.

9. The Apalachicola River is a slightly different problem. The stream has been regulated by upstream reservoirs, so that generally the flow will not drop below about 9600 cfs except when Jim Woodruff Dam may be shut down to attempt peaking operations. However, the aim is to provide 9600 cfs whenever there is any navigation on the river. This, of course, looks small to Committee members who represent large streams, but large or small, the problems are similar. The Apalachicola is relatively wide for this flow and its slope is about 0.40 ft per mile. While the low-water width varies from about 300 ft to over 800 ft, a substantial portion of the river is over 500 ft wide. As might be expected, these wide reaches present the most troublesome shoaling problems. In most places the banks of the river are essentially vertical and the entire width between banks is covered with water even during minimum flow. Thus, with few exceptions, the point bars and crossing shoals are submerged at all times. This condition may be observed in fig. 4. There is a large amount of bed load material underwater which is available for causing shoals. In addition the banks in a number of places are eroding moderately and furnishing shoal material (see fig. 5). Experience indicates that without contraction works, it would almost be impracticable to maintain, for any large portion of time, a channel with the authorized dimensions; therefore, construction of contraction works is about to be instituted. Initial works will contract the river to a width of about 500 ft by means of dikes. Pile dikes will be used wherever the overburden is of sufficient depth to support the piles. In the initial work, except for one longitudinal training dike, the dikes will be placed at right angles to the bank. The design of the



NOTE: QUARRY RUN STONE 200 LBS MAX SIZE

Fig. 3. Typical wood pile dike, Savannah River

Fig. 4. Apalachicola River

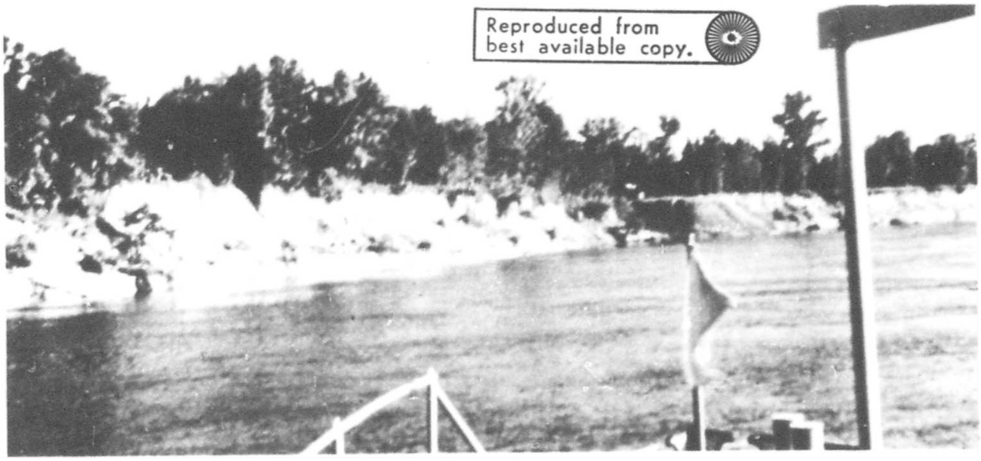


Fig. 5. Typical caving bank, Apalachicola River

dikes will be the same as used on the Savannah River except for minor modifications (fig. 6). The principal differences are the detached marker cluster and the shape of the blanket at the end of the dike.

10. The peninsula portion of the State of Florida is a considerably different problem. Practically all of it is erodible, consisting, as much of it does, of sand underlain by limerock. There are, however, large areas of muck and peat in such places as the Everglades area and along the upper St. Johns River. Even the muck and peat readily erode, so that when

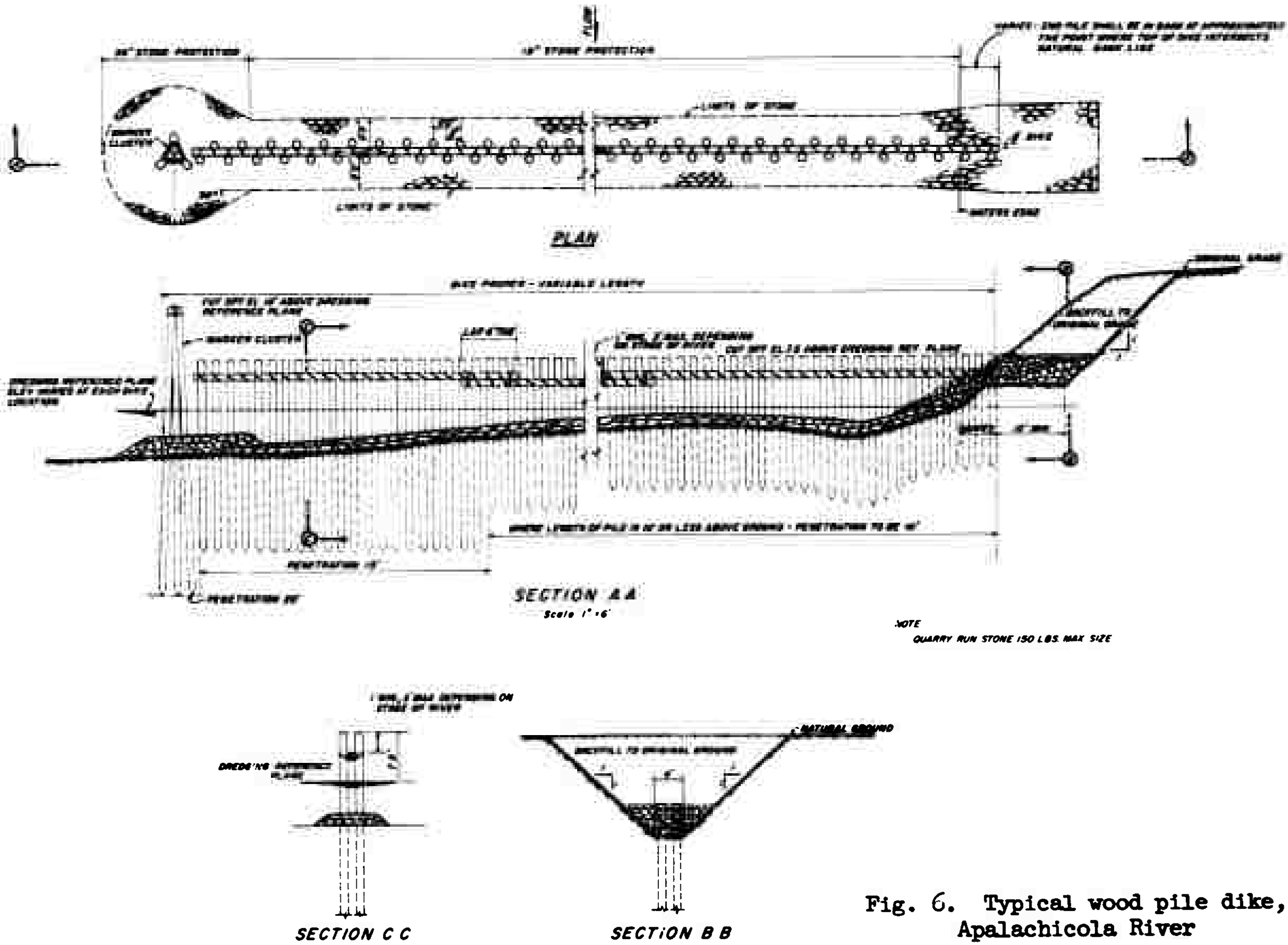


Fig. 6. Typical wood pile dike, Apalachicola River

channels are improved or canals are excavated, works must be constructed to limit the velocities. These usually are gated structures in the channels. The structures serve the dual purpose of minimizing erosion and preventing overdrainage. The latter is necessary, especially in southern Florida, because the rainfall there is seasonal and the main growing season is actually a dry season. Thus, water must be conserved and the water table held high enough to furnish moisture to the vegetation and to prevent oxidation of the peat lands.

11. Hydroelectric power plants that cause river stages to fluctuate rapidly and tend to increase bank erosion will continue to be built. Also, as this section of the country develops and the population increases, more improvements will be constructed near the streams and the property will become more valuable. Floodplain zoning may help, but it cannot stop eventual development as the need for land and access to the water grow. Thus it can be anticipated that within the South Atlantic Division there will be a growing demand for channel stabilization in order to prevent the rivers from destroying improvements and valuable property. Moreover, as additional rivers are improved for navigation, the need for bank stabilization and contraction works to provide authorized navigable depths and to minimize maintenance costs will multiply. Thus, it can be expected that there will be more and more construction of channel stabilization works throughout the Southeast as the years go by.

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2. U. S. Army Engineer District, Savannah, CE, Plans and Specifications for Contraction Works, Savannah River Below Augusta. Savannah, Georgia.

3. Wall, W. J., "Stabilization works on the Savannah River." Proceedings, American Society of Civil Engineers, vol 88, WW-1, New York, N. Y. (February 1962), pp 101-116.

CHAPTER III

STABILIZATION PLAN FOR THE ARKANSAS RIVER

by

E. B. Madden*

Introduction

1. The program for stabilization of the Arkansas River is but one element of a multiple-purpose plan of development for that part of the Arkansas River and its tributaries in Arkansas and eastern Oklahoma.

2. The multiple-purpose project provides for the improvement of the Arkansas River and tributaries in Arkansas and Oklahoma by construction of coordinated developments to serve navigation, produce hydroelectric power, afford additional flood control, and provide related benefits. The plan of development is shown in fig. 1. Included in the plan are a number of storage reservoirs which serve or will serve variously for flood control, hydroelectric-power generation, water supply for navigation, domestic and industrial water supply, and sediment detention as indicated on the map.

3. The navigation feature of the plan provides for a channel 9 ft deep following the Verdigris River from Catoosa, Oklahoma (near Tulsa), downstream to the Arkansas River, thence along the Arkansas River to the vicinity of Arkansas Post, Arkansas, from which point the route will follow a canal to the White River, then down the White River to the Mississippi River. The waterway will be canalized throughout its length of approximately 450 miles by a series of locks and dams. Four of the navigation dams on the main stem will have moderately high heads ranging from 30 to 50 ft. Two of these will include provisions for hydroelectric-power generation initially, and the other two will be so designed as to permit possible future addition of power facilities. The remaining navigation locks and dams will be relatively low-lift structures (generally 20 ft or less) with pools confined within the river channel. Bank stabilization and channel rectification of that part of the Arkansas River between Short

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* U. S. Army Engineer Division, Southwestern, Dallas, Texas.

Mountain Dam site and a point about 40 miles above the mouth of the river are also included as an important and, as will be shown later, necessary part of the multiple-purpose project.

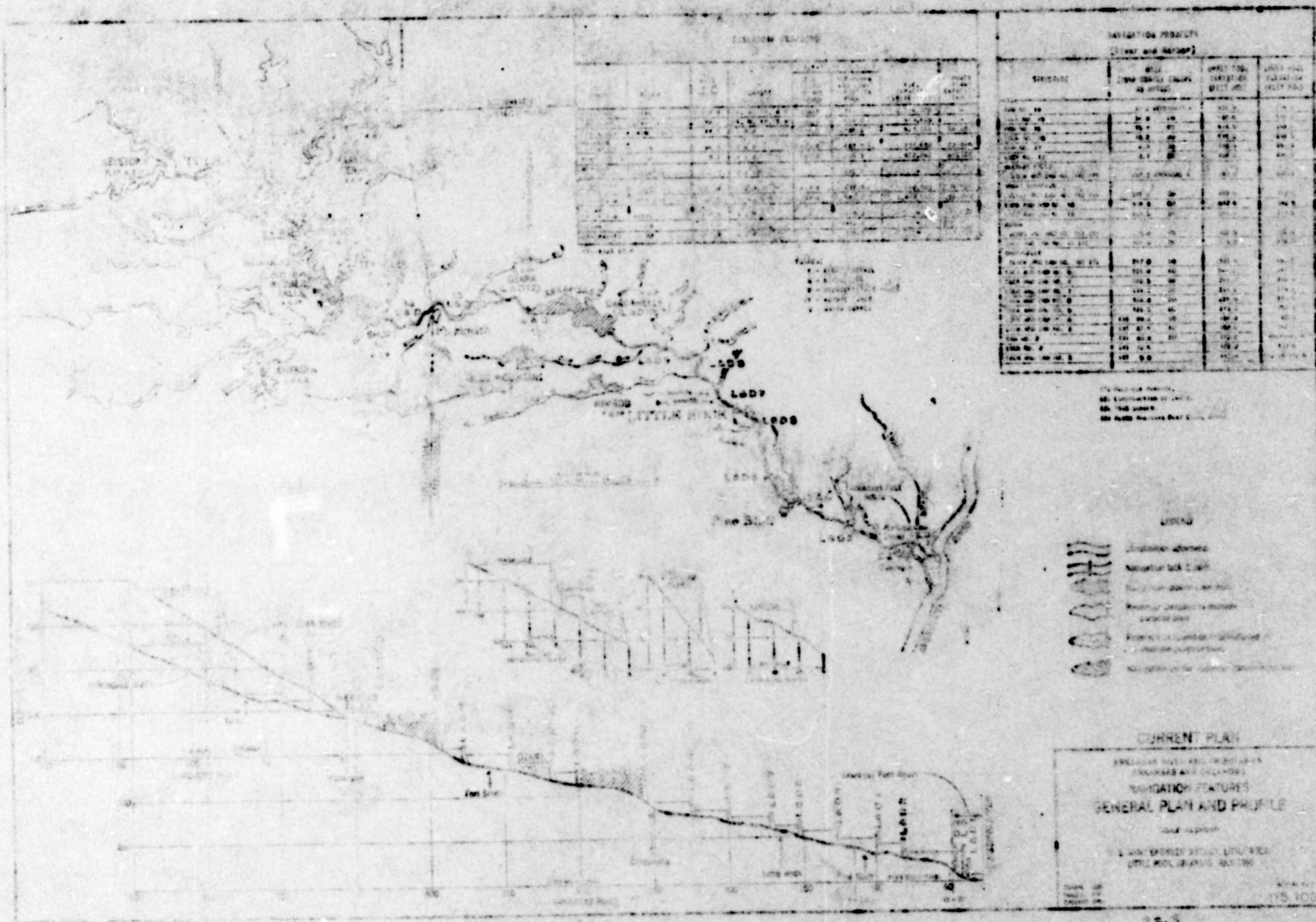
4. Stabilization work is not required in the downstream portions of the larger main-stem reservoirs where pools will be maintained and relatively slack-water conditions will exist at all flows. However, in the within-channel pools of the low-lift navigation locks and dams, essentially open-river conditions will prevail during the higher flows, and bank stabilization will be necessary in these pools as under natural conditions.

Stream characteristics

5. Between the Short Mountain Dam site and Little Rock, the Arkansas River flows through a mountainous area. Stream slopes vary from 1.0 ft to 0.8 ft per mile. The alluvial floodplain in this area varies from about one-half mile to about 6 miles in width and is bounded by shale and sandstone escarpments. Bedrock is from 5 to 30 ft below the thalweg of the river during low-water periods. Ample supplies of stone are available from existing or potential quarries near the river throughout the area upstream from Little Rock. This is a factor of considerable importance as stone is the principal construction material used in the stabilization structures.

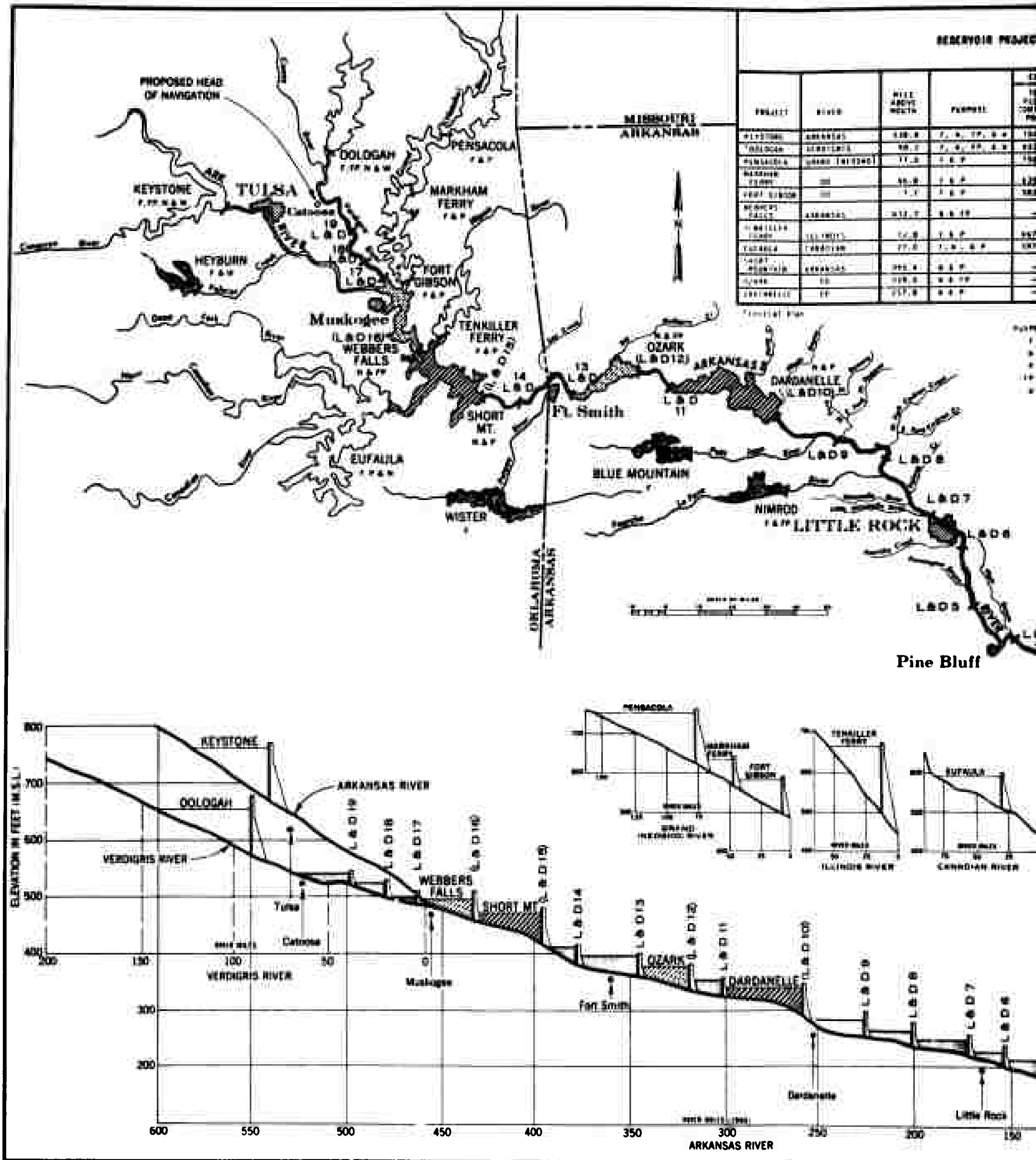
6. Immediately downstream from Little Rock the river enters the Mississippi River Alluvial Valley, and the Arkansas River alluvial floodplain is limited only along its south side between Little Rock and Pine Bluff by Eocene marine clay bluffs. Slopes vary from 0.8 to 0.6 ft per mile. Depth to bedrock increases downstream from Little Rock, exceeding 100 ft downstream from Pine Bluff. With the exception of a few areas where resistant marine clay is present in the streambed, the depth of scour is limited only by the erosive capacity of the stream itself and amounts to more than 50 ft below low-water level at some places. All stone used for stabilization works between Little Rock and the mouth of the river must come from quarries in the vicinity of Little Rock. As a result, the cost of stone is considerably greater in the lower part of the river than it is in the vicinity of Little Rock and upstream from Little Rock.

7. The Arkansas River carries a large sediment load, amounting to an average of about 100,000,000 tons per year at Dardanelle under natural conditions. It is estimated that the sediment outflow from Dardanelle



CURRENT PLAN
NAVIGATION FEATURES
GENERAL PLAN AND PROFILE
LITTLE ROCK

23-3


PROPOSED HEAD OF NAVIGATION
MISSOURI
ARKANSAS
DOLOGAH F, FP N & W
PENSACOLA F & P
KEYSTONE F, FP N & W
TULSA
Catoosa
19 L & D
18 L & D
17 L & D
MARKHAM FERRY F & P
FORT GIBSON F & P
HEYBURN F & W
Muskogee
(L & D 16) WEBBERS FALLS N & FP
TENKILLER FERRY F & P
ARKANSAS RIVER
14 L & D
13 L & D
OZARK (L & D 12)
ARKANSAS
DARDANELLE (L & D 10)
L & D 11
SHORT MT. N & F
Ft. Smith
EUFAULA F, P & W
BLUE MOUNTAIN
L & D 9
L & D 8
L & D 7
WISTER
NIMROD F & P
LITTLE ROCK
L & D 6
L & D 5
OKLAHOMA
ARKANSAS
Pine Bluff
RESERVOIR PROJECTS
KEYSTONE
DOLOGAH
ARKANSAS RIVER
VERDIGRIS RIVER
ELEVATION IN FEET (M.S.L.)
Tulsa
Catoosa
L & D 19
L & D 18
L & D 17
L & D 16
L & D 15
WEBBERS FALLS
SHORT MT.
L & D 14
L & D 13
L & D 12
L & D 11
L & D 10
L & D 9
L & D 8
L & D 7
L & D 6
OZARK
DARDANELLE
Muskogee
Fort Smith
Dardanelle
Little Rock
PENSACOLA
MARKHAM FERRY
FORT GIBSON
TENKILLER FERRY
EUFAULA
ILLINOIS RIVER
CANADIAN RIVER
ARKANSAS RIVER

23.1

23.2

RESERVOIR PROJECTS

PROJECT	RIVER	MILE ABOVE MOUTH	PURPOSE	ELEVATION (M.S.L.) TOP FLOOD CONTROL POOL	ELEVATION (M.S.L.) TOP POWER AND CONSERVATION POOL	F.C. CAPACITY (ACRE-FEET)	POWER PLANT CAPACITY (KW.)
KEYSTONE	ARKANSAS	538.8	F, N, FP, & W	754.0	714.5	1,414,000	-
*OOLOGAH	VERDIGRIS	90.2	F, N, FP, & W	651.0	608.0	963,000	-
PENSACOLA	GRAND (NEOSHO)	77.0	F & P	756.1	746.1	523,000	86,400
MARKHAM FERRY	DO	46.8	F & P	634.0	617.0	233,000	72,000
FORT GIBSON	DO	7.7	F & P	582.0	554.0	922,000	67,500
WEBBERS FALLS	ARKANSAS	432.3	N & FP	-	490.0	-	-
TENKILLER FERRY	ILLINOIS	12.8	F & P	667.0	630.0	600,000	34,000
EUFAULA	CANADIAN	27.0	F, N, & P	597.0	585.0	1,470,000	90,000
SHORT MOUNTAIN	ARKANSAS	395.4	N & P	-	460.0	-	110,000
OZARK	DO	319.0	N & FP	-	372.0	-	-
DARDANELLE	DO	257.8	N & P	-	338.0	-	124,000

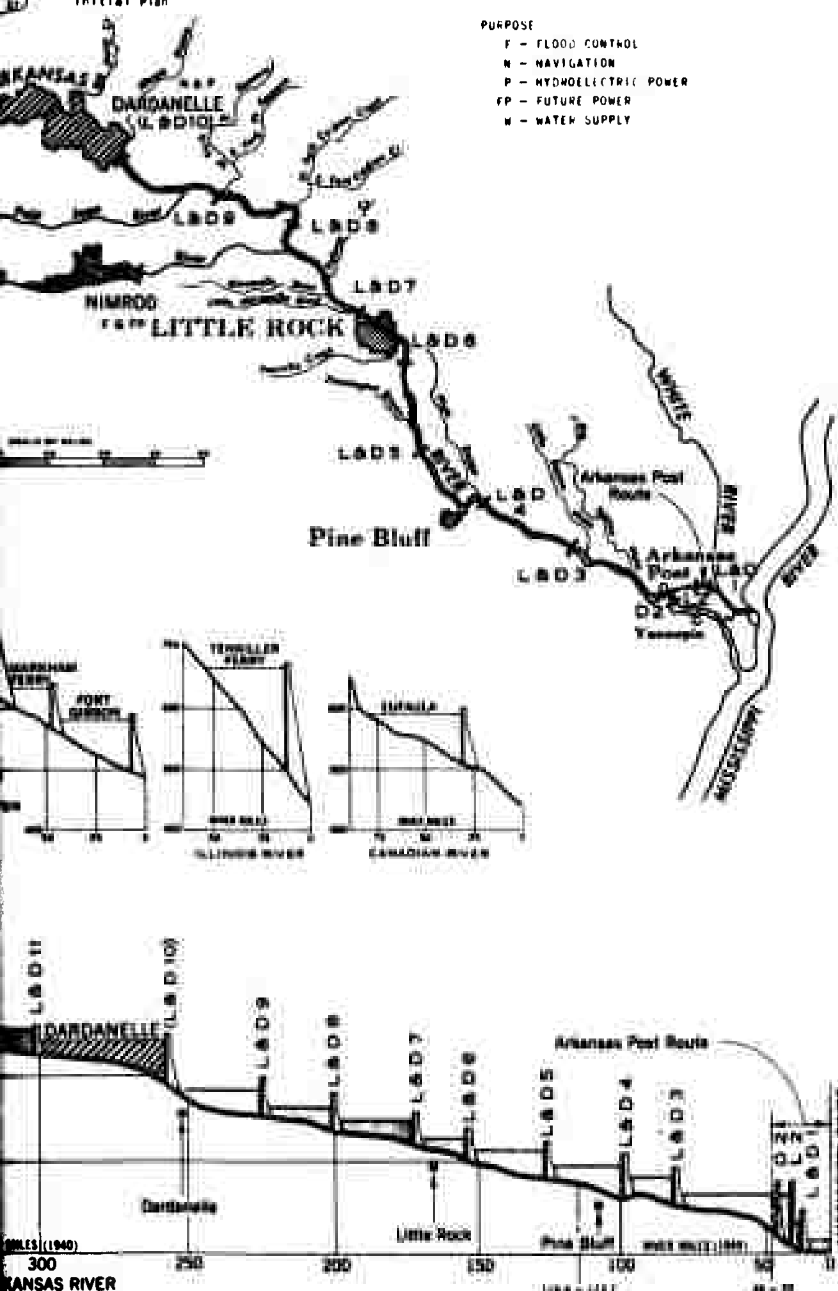
*Initial Plan

PURPOSE
- F - FLOOD CONTROL
- N - NAVIGATION
- P - HYDROELECTRIC POWER
- FP - FUTURE POWER
- W - WATER SUPPLY

NAVIGATION PROJECTS
(River and Harbor)

STRUCTURE	MILE (1940 SURVEY EXCEPT AS NOTED)	UPPER POOL ELEVATION (FEET MSL)	LOWER POOL ELEVATION (FEET MSL)
DAM NO. 19	37.2 VERDIGRIS R.	532.0	515.0
LOCK NO. 19	36.9 DO	532.0	515.0
DAM NO. 18	19.4 DO	515.0	498.0
LOCK NO. 18	19.2 DO	515.0	498.0
DAM NO. 17	4.0 DO	498.0	490.0
LOCK NO. 17	3.7 DO	498.0	490.0
WEBBERS FALLS (LOCK AND DAM NO. 16) (1)	432.3 ARKANSAS R.	490.0	460.0
SHORT MOUNTAIN (LOCK AND DAM NO. 15) (1)	395.4 DO	460.0	412.0
LOCK AND DAM NO. 14	379.1 DO	412.0	392.0
LOCK AND DAM NO. 13	346.5 DO	392.0	372.0
OZARK (LOCK AND DAM NO. 12) (1)	319.0 DO	372.0	348.0
LOCK AND DAM NO. 11 (2)	301.8 DO	348.0	338.0
DARDANELLE (LOCK AND DAM NO. 10) (1)	257.8 DO	338.0	284.0
LOCK AND DAM NO. 9	225.0 DO	284.0	265.0
LOCK AND DAM NO. 8	200.5 DO	265.0	249.0
LOCK AND DAM NO. 7	171.6 DO	249.0	229.0
LOCK AND DAM NO. 6	154.4 DO	229.0	213.0
LOCK AND DAM NO. 5	128.0 DO	213.0	196.0
LOCK AND DAM NO. 4	(3) 99.8 DO	196.0	182.0
LOCK AND DAM NO. 3	(3) 82.5 DO	182.0	162.0
DAM NO. 2	(3) 46.8 DO	162.0	ARKANSAS R.
LOCK NO. 2	(4) 12.5	162.0	137.0
LOCK AND DAM NO. 1	(4) 9.8	137.0	112 (WHITE R.)

(1) Multiple purpose.
(2) Construction deferred.
(3) 1949 Survey.
(4) Along Arkansas Post Canal Route.



LEGEND

- Canalization approved
- Navigation lock & dam
- Navigation-power reservoir
- Reservoir included in multiple-purpose plan
- Reservoir in operation (not included in multiple-purpose plan)
- Navigation-power reservoir (power deferred)

CURRENT PLAN

ARKANSAS RIVER AND TRIBUTARIES
ARKANSAS AND OKLAHOMA

NAVIGATION FEATURES

GENERAL PLAN AND PROFILE

SCALE: AS SHOWN

U. S. ARMY ENGINEER DISTRICT, LITTLE ROCK
LITTLE ROCK, ARKANSAS, JULY 1960

DRAWN HJH
TRACED HJH
CHECKED JWW

SERIAL 6238
115/108

23.2

23.3

Fig. 1

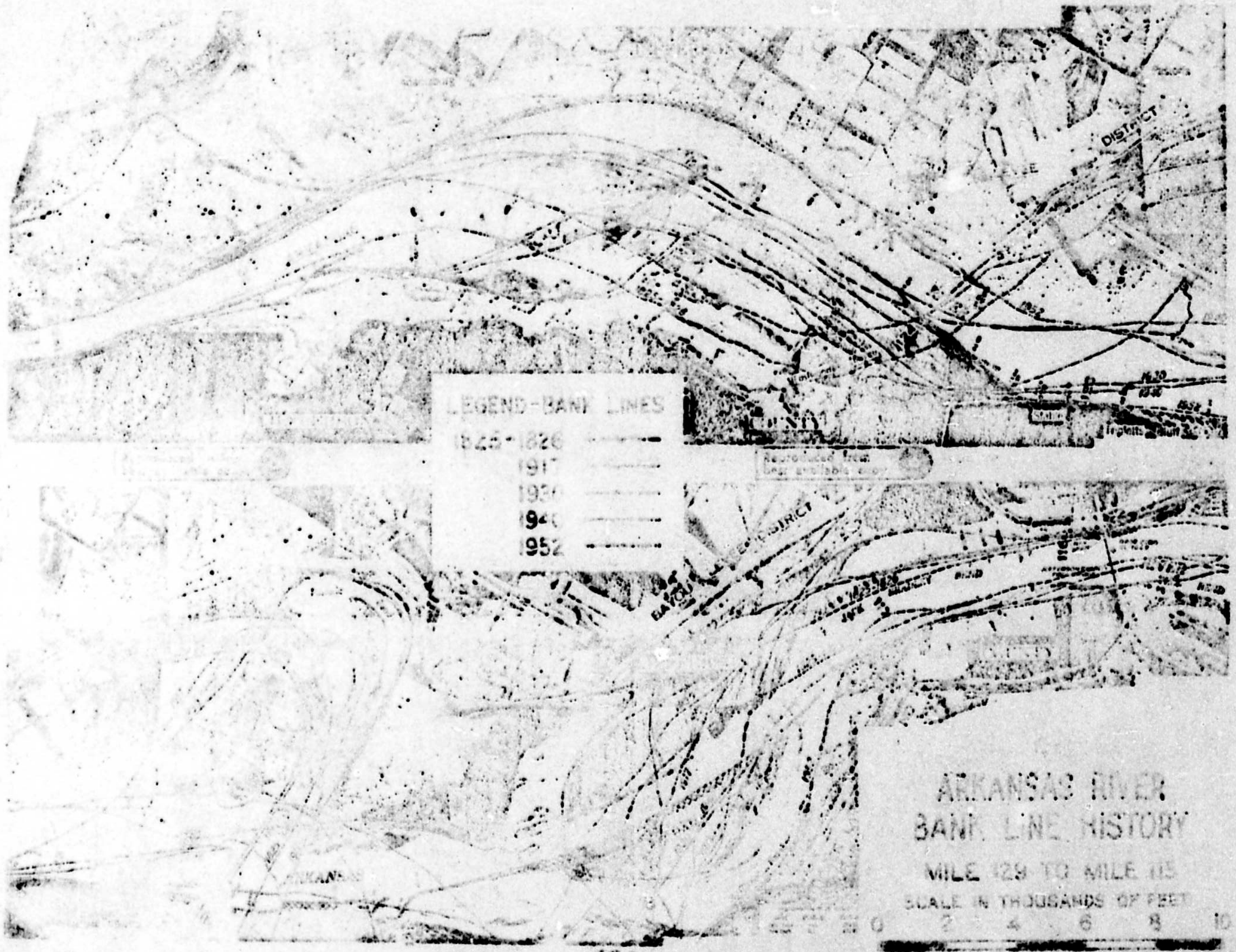
Reservoir will be reduced to about 10,000,000 tons per year after closure of Dardanelle and the large dams upstream. As is characteristic of such heavily laden streams, except at a few localities where the river flows for short stretches against rock bluffs upstream from Little Rock and against stable clay bluffs at two spots between Little Rock and Pine Bluff, the river channel is subject to extensive bank caving and shifting through its alluvium of sand, silt, and clay. Some idea of the nature of this channel shifting is illustrated by fig. 2, on which is shown the bank-line history of a typical stretch of river. The bank lines shown cover the period from 1917 through 1952. Bank lines of 1825-1826 are also shown for comparison. The mosaic map on which the bank lines are superimposed was made from aerial photographs taken in 1958. Attention is invited to the levees shown on the map. Several levee setbacks have been necessary in this area in the past in order to prevent destruction of the levees, and additional setbacks of these levees were imminent at the time the worst of the bank caving was brought under control in 1950 to 1952 by bank stabilization work in this area. This is typical of experience at nearly all the numerous levee projects which have been built along the river. In addition, caving banks have damaged or are threatening to damage numerous water, oil, and gas pipeline crossings and have threatened to cause flanking of a number of bridges such as the one shown on this map. Large areas of productive farmland are lost each year. Bank recession of 200 to 300 ft during a single rise is a common occurrence, and recession of as much as 1200 ft during a single flood period has been experienced. All this points up the necessity of bank stabilization and channel rectification work in order to obtain a stable channel suitable for navigation, to prevent flanking of the locks and dams and of bridges, and to prevent the continuing destruction or setting back of levees, damages to pipeline crossings, and loss of productive farmland and buildings.

Stabilization plan

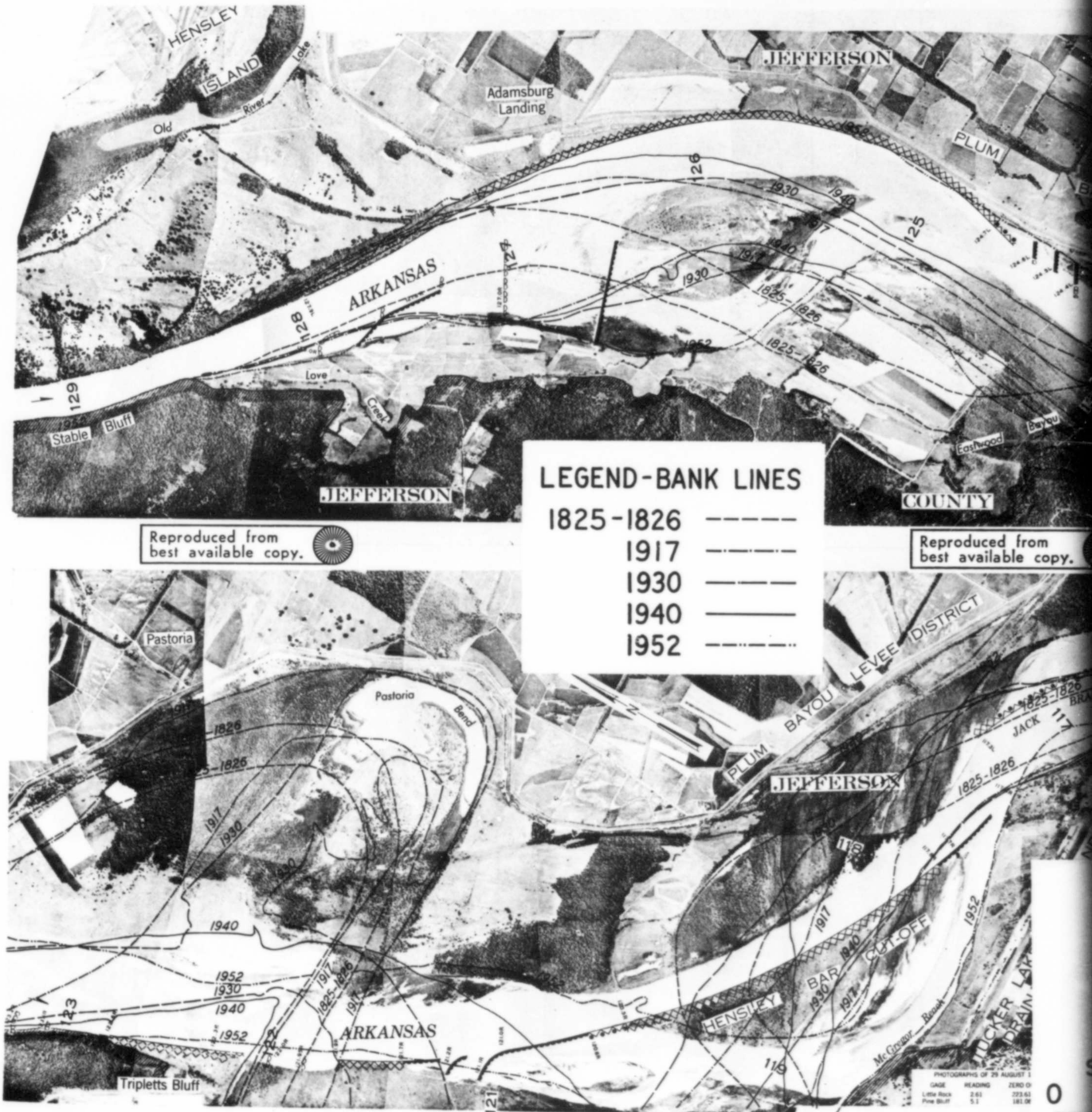
8. General features. The general procedure being used for stabilizing the Arkansas River involves shaping the stream into one channel along an alignment consisting of a series of easy bends and permanently fixing the channel on that alignment. The policy is to control the river by guiding or leading it rather than by attempting to resist it forcibly.

A typical layout of the various types of bank stabilization and channel rectification structures being used to carry out the plan of development is shown in fig. 3 which is a map covering the same area shown on the bank-line history map, fig. 2. Here are illustrated the use of a cutoff channel to eliminate two sharp bends; the use of revetments to stabilize the concave banks of bends; the use of dikes to close off a secondary channel, to control the direction of flow from one bend to another, and to "fair out" the concave banks of bends along a more desirable alignment. As used herein the term "revetment" applies to all structures parallel to the current of the realigned channel, and the term "dikes" applies to structures at an angle to the current. Revetment types include standard revetment with or without mattress, which is used on banks of sufficient height to support these types, and pile revetment or stone-fill revetment, which is used at locations where the desired new channel control line lies in the existing river channel or across low sandbars. Dikes may be either pile structures or stone-fill structures, and in some special cases, steel jetties are used. The latter type is not illustrated on the typical layout map, but is considered advantageous for use in some cases for supplementing chute closure structures and on low bars along the convex side of the channel whenever it is desired to induce more accretion deposits. Details of the various types of structures will be discussed later after consideration of channel alignment.

9. Alignment. Shaping the river into a proper alignment consisting of a series of easy bends with the flow directed from one bend into the next in such a way as to maintain a direction essentially parallel to the channel control line is considered of paramount importance in the interest of producing the minimum practical attack on the stabilization structures and thus assuring an economical development. One of the essential requirements for maintaining such an alignment is that the stabilization work in any section of the river be started at some stabilized point upstream and worked progressively downstream to some other stable location or to some point beyond which the river can be safely left uncontrolled. Construction of isolated stabilization works has proven unsuccessful again and again. Changes in the direction of flow within the uncontrolled section either will eventually set up a direct attack against such isolated protective

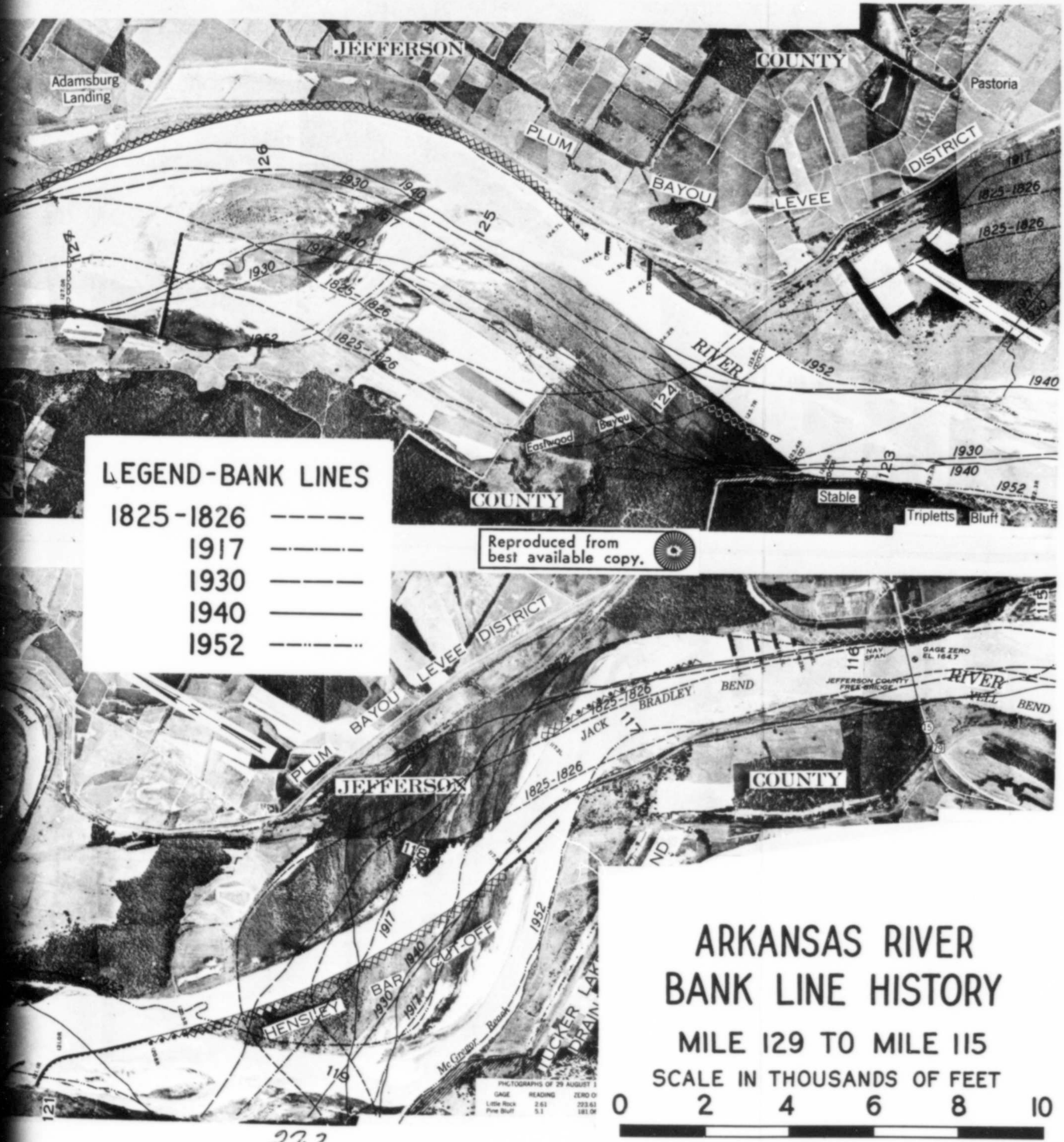


LEGEND-BANK LINES
1826
1917
1930
1940
1952
DISTRICT
ARKANSAS RIVER
BANK LINE HISTORY
MILE 129 TO MILE 115
SCALE IN THOUSANDS OF FEET
0 2 4 6 8 10


HENSLEY
ISLAND
Old
River
Lake
Adamsburg
Landing
JEFFERSON
PLUM
ARKANSAS
129
128
127
126
125
Stable Bluff
Love
Creek
JEFFERSON
COUNTY
Eastwood
Bayou
LEGEND-BANK LINES
1825-1826
1917
1930
1940
1952
Pastoria
Pastoria
Bend
PLUM BAYOU LEVEE DISTRICT
JEFFERSON
JACK
117
118
HENSLEY BAR CUT-OFF
McGregor Reach
TUCKER LAKE DRAIN
ARKANSAS
123
122
121
119
Tripletts Bluff
PHOTOGRAPHS OF 29 AUGUST
GAGE
READING
Little Rock
2.61
Pine Bluff
5.1

27.1

27.2

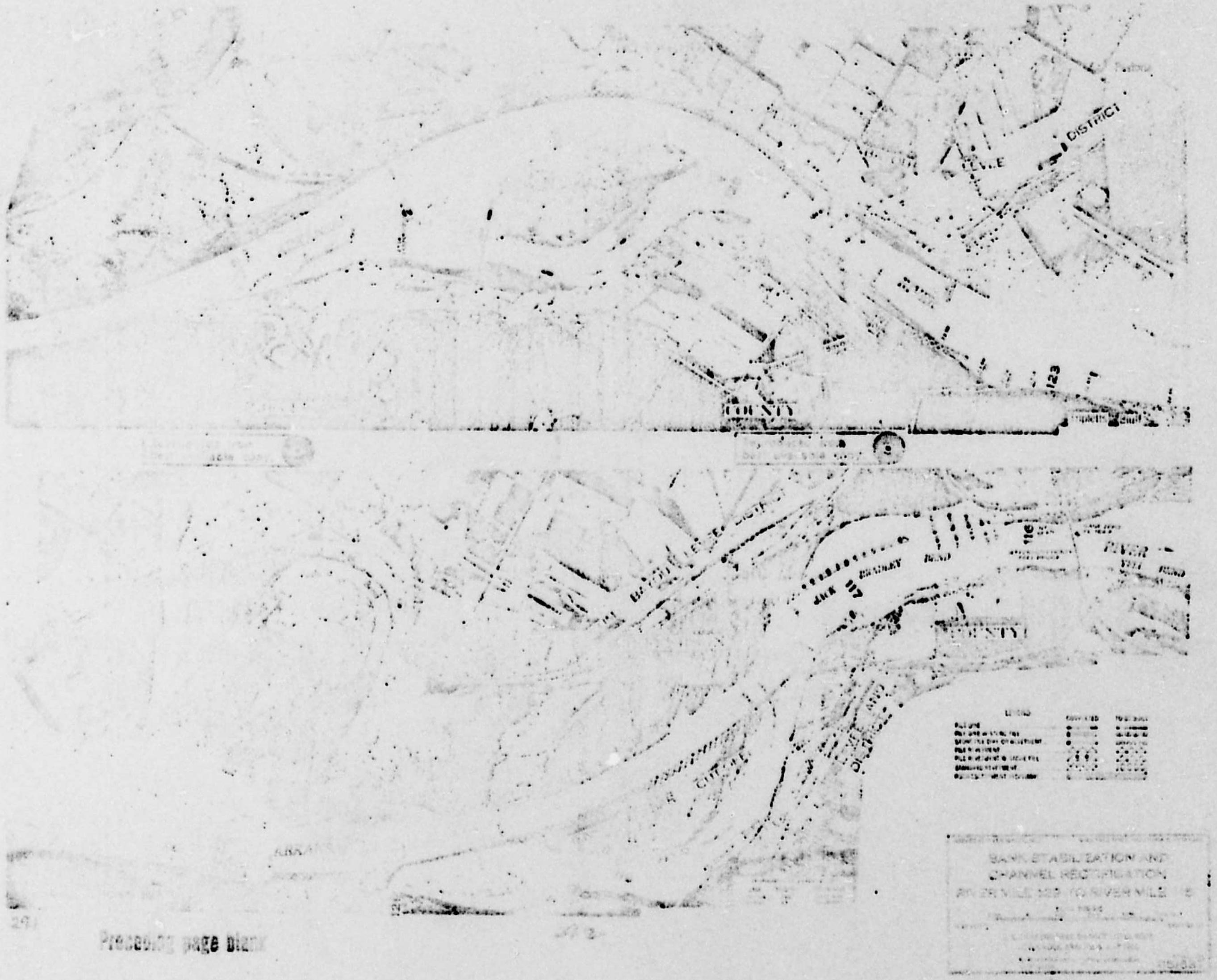


JEFFERSON
COUNTY
Adamsburg Landing
Pastoria
PLUM
BAYOU
LEVEE
DISTRICT
RIVER
Eastwood Bayou
COUNTY
Stable
Tripletts Bluff
LEGEND-BANK LINES
1825-1826
1917
1930
1940
1952
Reproduced from best available copy.
PLUM BAYOU LEVEE DISTRICT
JEFFERSON
COUNTY
JACK BRADLEY BEND
RIVER
BEND
JEFFERSON COUNTY FREE BRIDGE
HENSLEY BAR CUT-OFF
McGregor Bench
ARKANSAS RIVER
BANK LINE HISTORY
MILE 129 TO MILE 115
SCALE IN THOUSANDS OF FEET
0 2 4 6 8 10

27.2

27.3

Fig. 2



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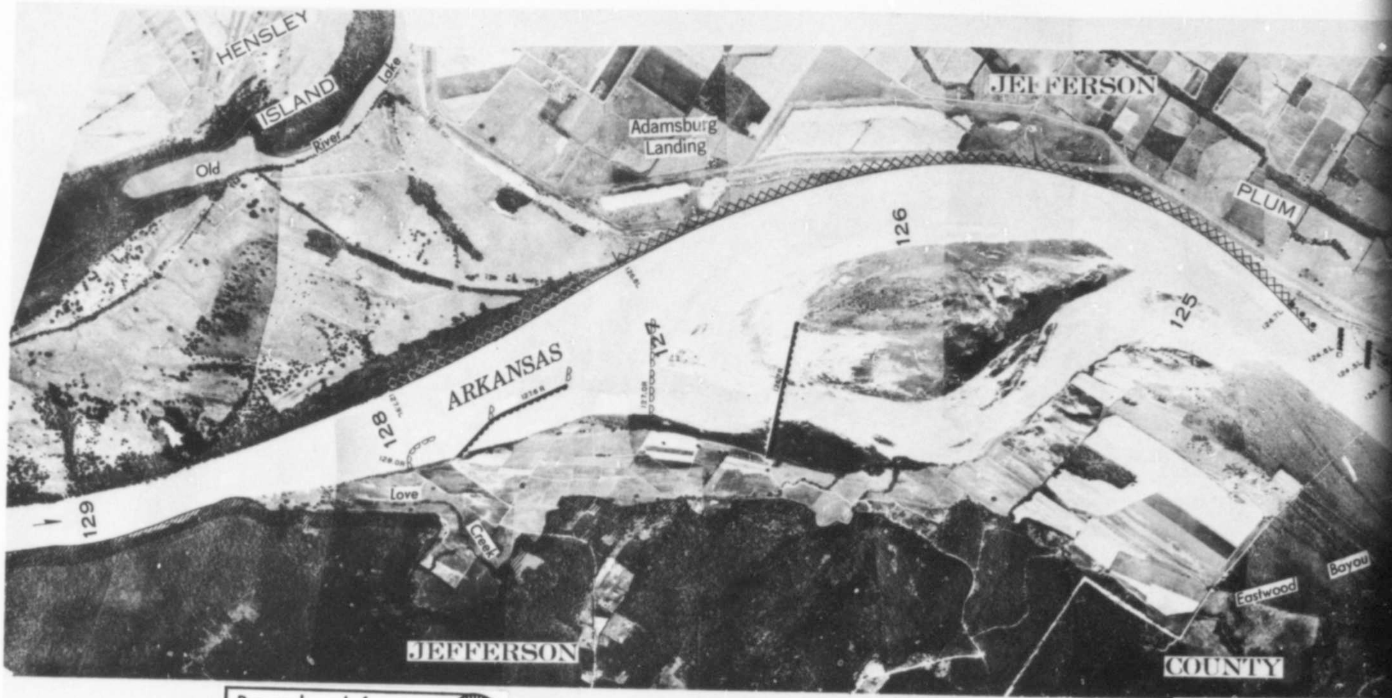

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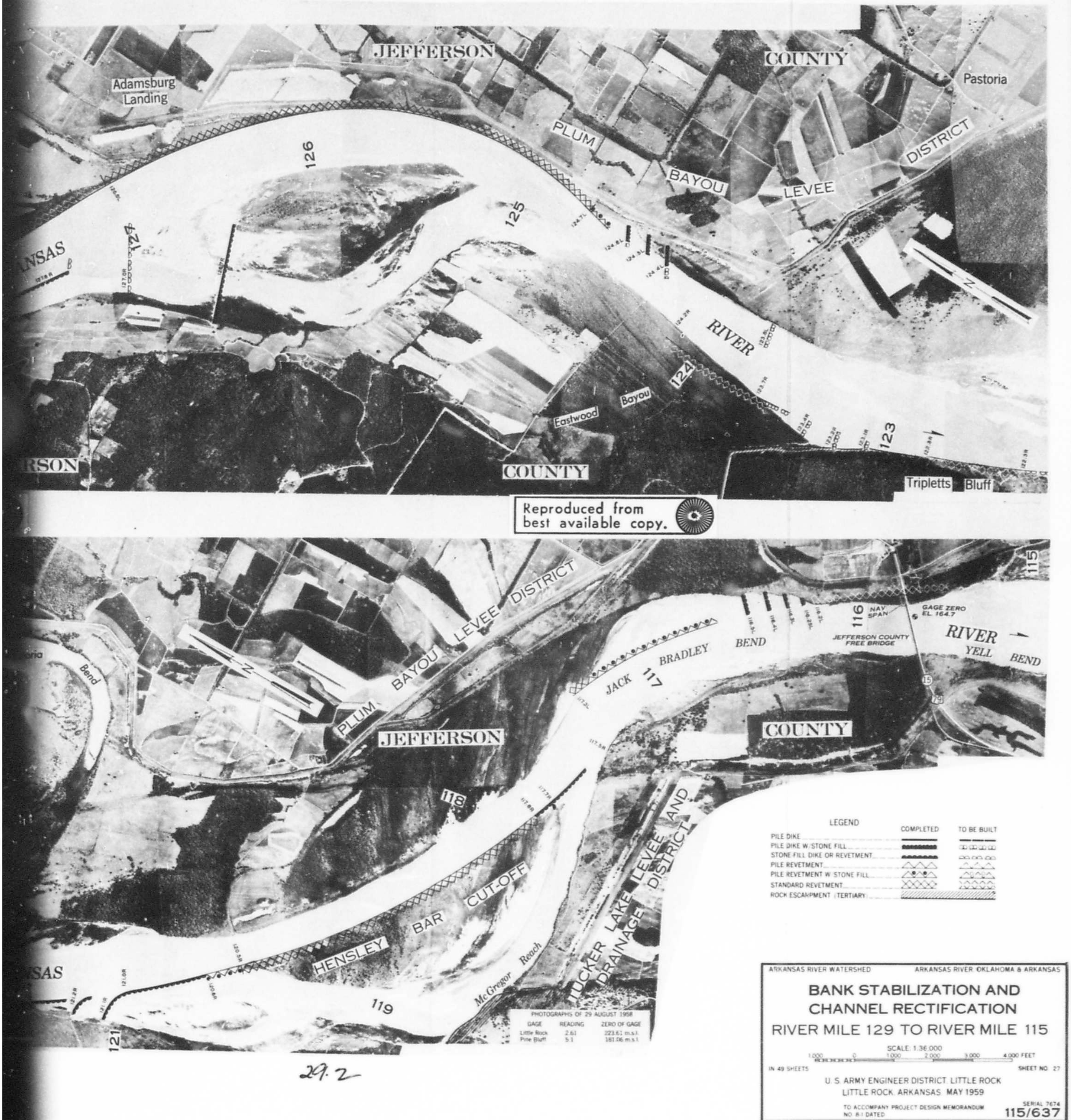


JEFFERSON
COUNTY
Adamsburg Landing
Pastoria
PLUM BAYOU LEVEE DISTRICT
126
125
124
123
RIVER
Eastwood Bayou
COUNTY
Tripletts Bluff
JACK BRADLEY BEND
117
116
118
119
121
115
NAV SPAN
GAGE ZERO EL 164.7
JEFFERSON COUNTY FREE BRIDGE
RIVER
YELL BEND
Bend
JEFFERSON
COUNTY
HENSLEY BAR CUT-OFF
McGregor Reach
TUCKER LAKE LEVEE AND DRAINAGE DISTRICT
PHOTOGRAPHS OF 29 AUGUST 1958
LEGEND
COMPLETED
TO BE BUILT
PILE DIKE
PILE DIKE W/STONE FILL
STONE FILL DIKE OR REVETMENT
PILE REVETMENT
PILE REVETMENT W/STONE FILL
STANDARD REVETMENT
ROCK ESCARPMENT (TERTIARY)
ARKANSAS RIVER WATERSHED
ARKANSAS RIVER OKLAHOMA & ARKANSAS
BANK STABILIZATION AND CHANNEL RECTIFICATION
RIVER MILE 129 TO RIVER MILE 115
SCALE: 1:36,000
1,000 0 1,000 2,000 3,000 4,000 FEET
IN 49 SHEETS
SHEET NO. 27
U. S. ARMY ENGINEER DISTRICT, LITTLE ROCK
LITTLE ROCK, ARKANSAS, MAY 1959
TO ACCOMPANY PROJECT DESIGN MEMORANDUM NO. 8-1 DATED
SERIAL 7674
115/637

29.2

29.3

Fig. 3

works, which in turn may severely damage or destroy them, or will shift the attack to some other point, thus requiring additional work and possible abandonment of the existing works.

10. Appropriate attention to width of the channel, especially at crossings, is another essential element in assuring a suitable alignment of flow and minimum attack on structures downstream. On the basis of an examination of widths attained by naturally stable sections of the river, a design trace width ranging from 1000 to 1500 ft has been selected. It is pertinent to mention that special measures are proposed for contraction and dredging in the upper ends of the navigation pools in the interest of increasing depths so as to permit spacing dams farther apart than could otherwise be done. However, a discussion of this is beyond the scope of this paper. Model tests in a movable-bed channel model at the Waterways Experiment Station were included in the studies made in connection with the special measures for contraction and dredging.

11. Also in the interest of stability of the works, the practice of constructing revetments on a smooth alignment free of irregularities and false points is followed on the Arkansas River in order to avoid the localized scour and undermining of the revetment that is often produced by eddies set up by such disturbances to the flow.

12. Sharp curvature of bends is also to be avoided if economical control of the river is to be obtained. The shorter the radius of curvature of the bend, the deeper the channel will be adjacent to the concave bank. The deeper the channel is, the greater will be the possibility of undermining the protection works in stabilized bends and, hence, the greater will be the cost of maintaining the stabilization structures. This is illustrated by the chart, fig. 4. Included on this chart are curves of relation between radius of curvature and maximum depth in bends on the Arkansas and Missouri Rivers. (These two streams, by the way, are very similar in flow characteristics and in shape, and many of the criteria in use for the Arkansas River stabilization work were adopted on the basis of experience on the Missouri River and upon the advice of representatives of the Missouri River Division and the Kansas City and Omaha Districts of the Corps of Engineers.) The curves for both rivers indicate that the maximum depth increases sharply as the radius of curvature drops below about 7000

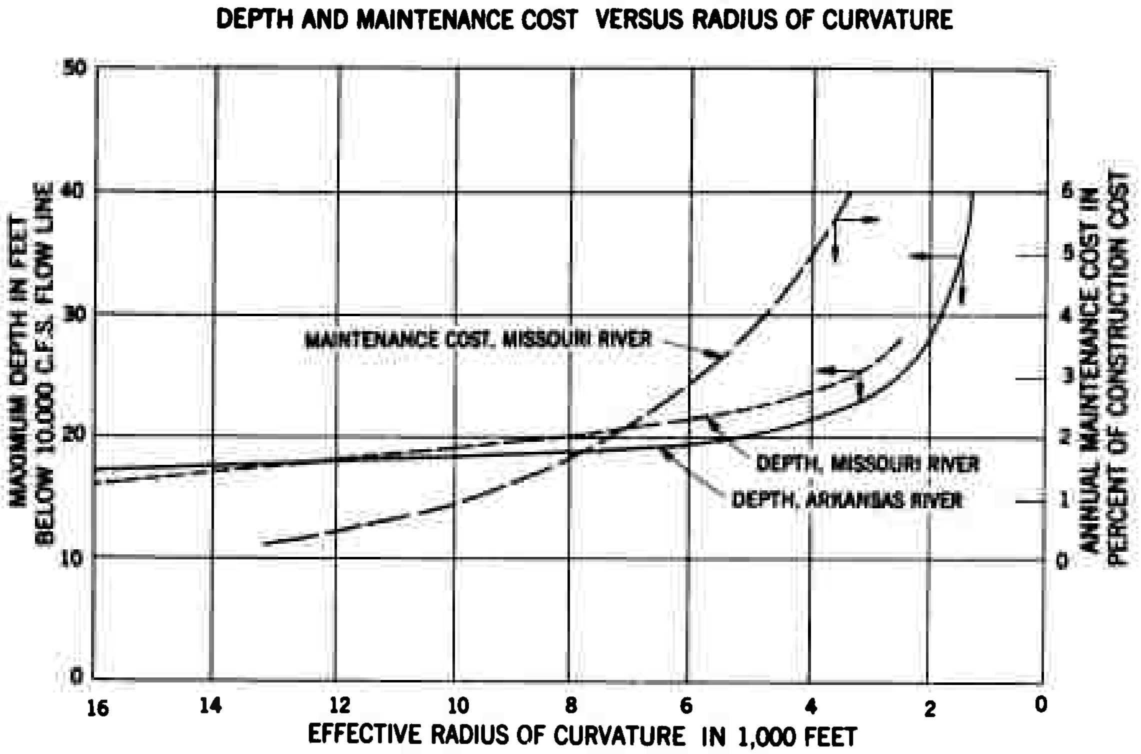


Fig. 4. Depth and maintenance cost versus radius of curvature

or 8000 ft. Also shown on the chart is a curve of relation between the radius of curvature and annual maintenance cost for standard revetments with mattress on the Missouri River. This curve also shows a sharp increase in cost with decrease in radius, the maintenance cost amounting to only about 1-1/2 percent of the construction cost for revetments with an effective radius of 8000 ft and rising to 5 percent for revetments with a radius of 4000 ft. Missouri River data were used for this illustration because of the longer record of experience on that river. However, the brief experience with maintenance of the Arkansas River bank stabilization works has agreed with the Missouri River experience, at least qualitatively. The only severe damage to revetment work on the Arkansas thus far has occurred at bends of short radii or where the effective radius of the approaching flow is short. A radius of 8000 ft is considered to be the desirable minimum for use on the Arkansas and most bends will have radii equal to or greater than this. However, shorter radius bends have been or will be included at a few locations where longer radius bends are precluded by existing physical conditions or where flow is directed against naturally stable banks. In addition to being desirable in the interest of stability of the

structures, bends with radii of 8000 ft or longer will also be more advantageous for movement of navigation tows than would sharper curves. At the other extreme, if the channel alignment is straight or of too flat a curvature, the flow will tend to shift from side to side, making it more difficult and expensive to control. Desirable upper limits of radii of curvature appear to be about 16,000 ft in the Short Mountain Dam site to Fort Smith area, and about 24,000 ft in the lower part of the river.

13. Stabilization structures. Let us now consider the details of the various types of structures used for controlling the river. In fig. 5 are shown a typical section and part plan of the familiar standard revetment with board mattress. This type of revetment with a minimum width of mattress of 85 ft and with ballast stone at the rate of 1 ton per square of 100 sq ft has been used to a limited extent upstream of the vicinity of Pine Bluff. This type is used principally downstream of the vicinity of Pine Bluff with a minimum mattress width of 101 ft and with 2 to 6 tons of stone per square on the mattress, depending on the sharpness of the bend,

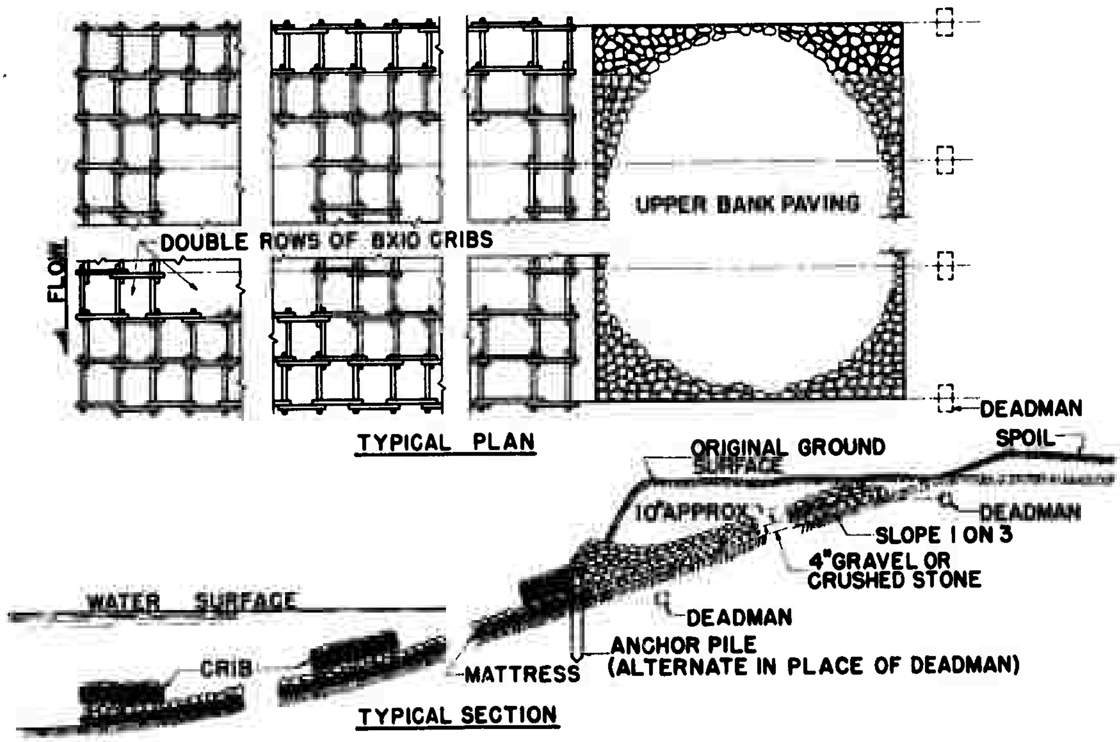


Fig. 5. Standard revetment with mattress, Arkansas River bank stabilization

the depth of the river, and the severity of the attack on the revetment.

14. In fig. 6 are shown typical sections of standard revetment without mattress, also known as a trench-fill revetment. This type was developed and used first on the Missouri River. It has proven successful after about 17 years of use on the Missouri and about 11 years use on that part of the Arkansas River upstream of the vicinity of Pine Bluff. The "Construction Reference Plane," or CRP, shown on these and subsequent illustrations is a sloping plane corresponding approximately to the mean low-water profile and is used as the reference for vertical control of the stabilization structures. Similarly, the "Structure Azimuth Line," or SAL, serves as the basis for horizontal control of the structures. Three variations in the section for the standard revetment without mattress are shown in the drawing. The top section applies to locations where the desired rectified channel line is some distance behind the existing river bank line. Here, stone toe-trench fill and stone upper-slope paving are placed, as shown, in a trench excavated to a depth of 7 or 8 ft below CRP. Specifications provide that dewatering equipment shall be used if necessary in order to obtain the desired depth of trench. It is important that the stone in the toe trench be placed to as low an elevation as practical. This type of section provides the desired underwater protection through the launching of a portion of the toe-trench fill after the bank line recedes to the revetment line, somewhat as indicated by dashed lines on the cross section. The strength of the underwater toe can be readily increased or decreased as required by modification of the depth or slope of the trench fill. The practical limit of channel depth for use of trench-fill revetment is considered to be about 40 ft below CRP if a sufficient quantity of stone is provided. For greater economy, however, it is considered advisable to shape the alignment of the bends so as to limit water depth to a maximum of 20 to 25 ft below CRP.

15. The middle section on this drawing applies to locations where the desired rectified channel line coincides approximately with the existing bank line. The bottom section is used where the desired channel line is slightly riverward of the existing bank line for a short distance. Here a stone fill is placed along the structure azimuth line, the area between the stone fill and the existing bank is filled in with a semicompacted earth

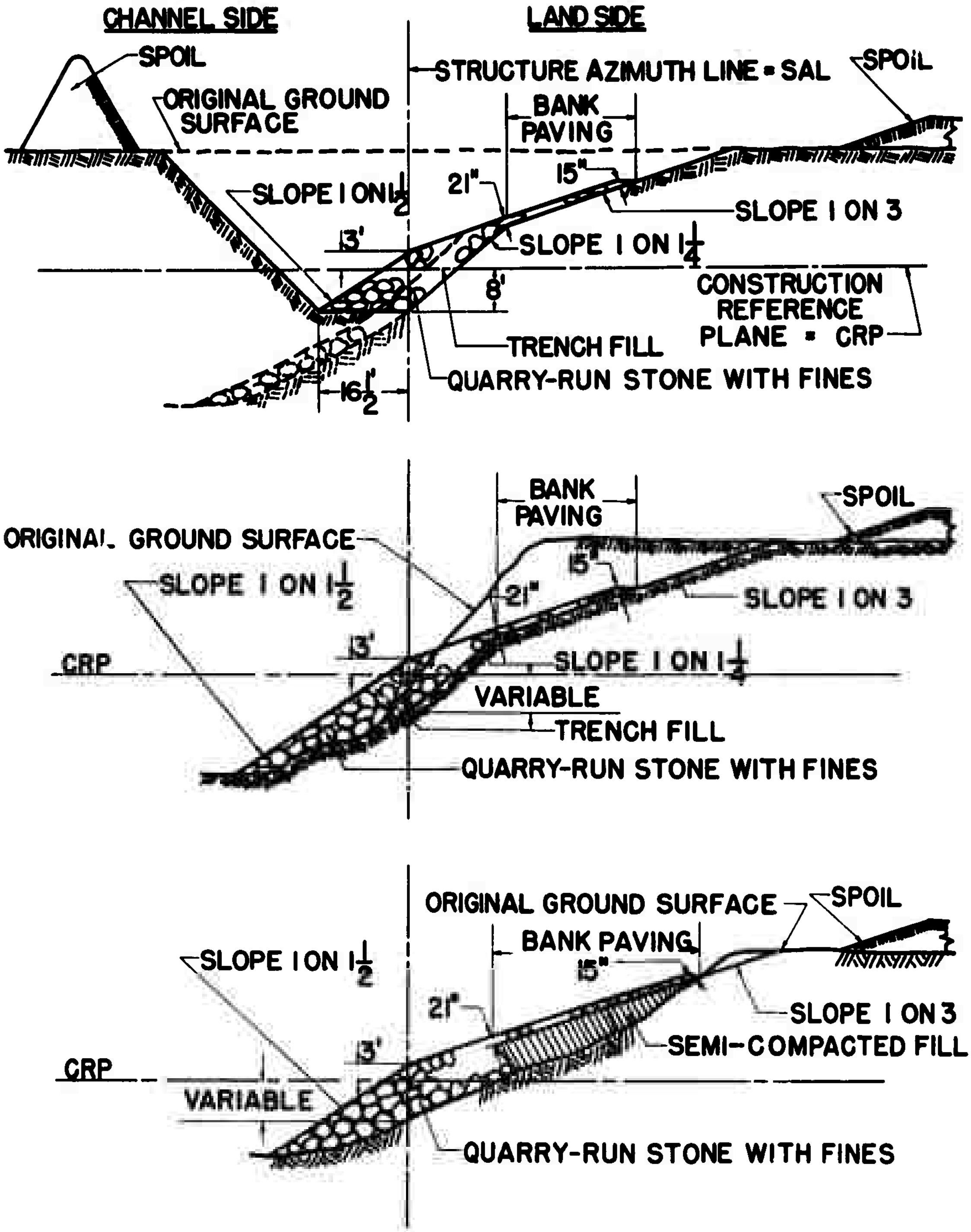


Fig. 6. Typical sections for standard revetment without mattress (trench fill), Arkansas River bank stabilization

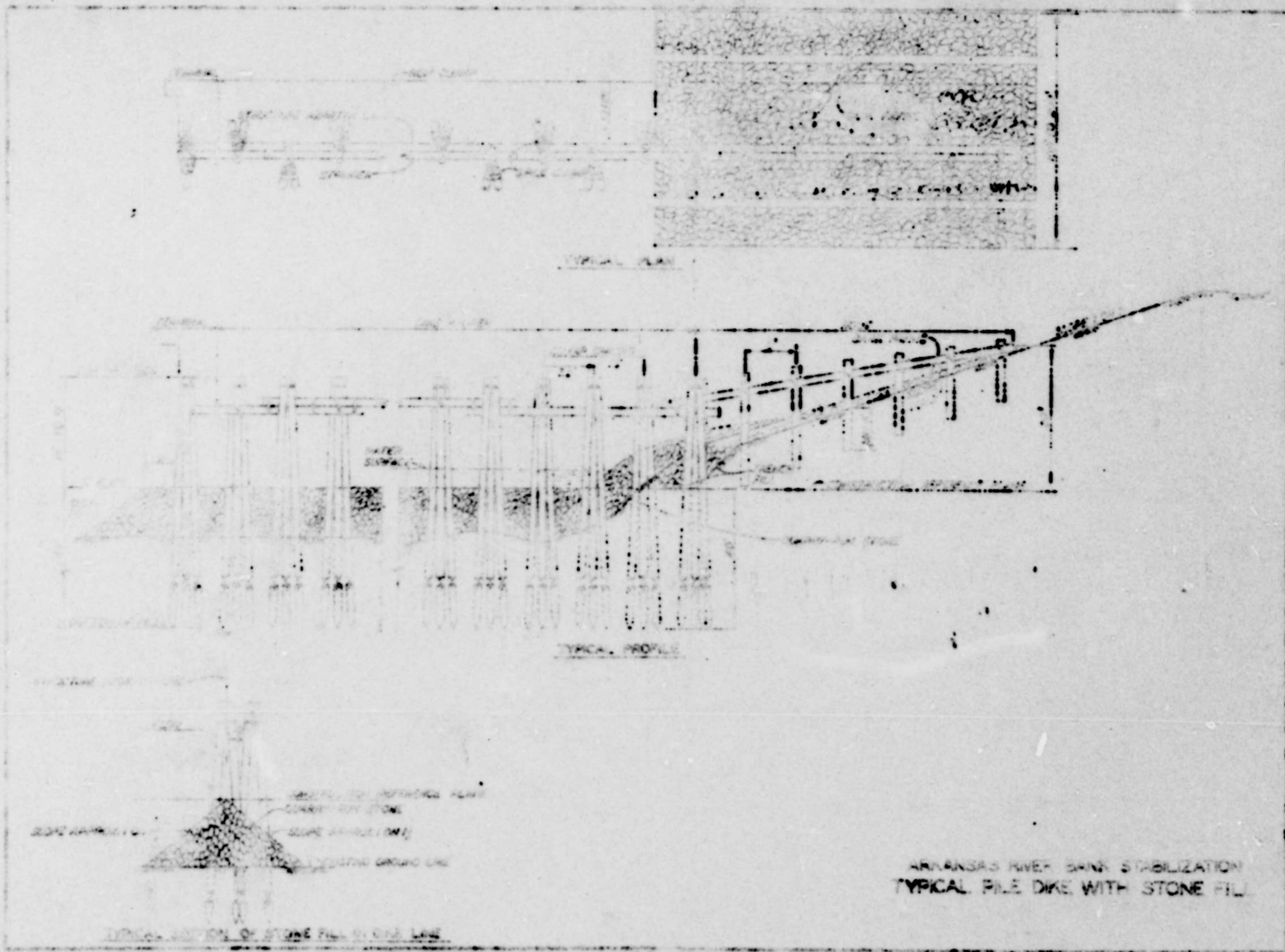
fill on a sloping plane, and the fill is protected by a layer of stone.

16. The upper bank paving of the standard revetment without mattress, as well as of revetment with mattress, is generally carried to a height of 16 ft above CRP in the areas upstream of the vicinity of Pine Bluff. The graded bank above the top of the paving, amounting to 5 to 15 ft in vertical height, and the riverward slope of the spoil bank are protected from erosion by planted grass. This has proven satisfactory in all except a few localities at which temporary obstructions to flow produced localized eddies and concentrations of currents that scoured the bank above the paving. Damage from this source has been relatively minor.

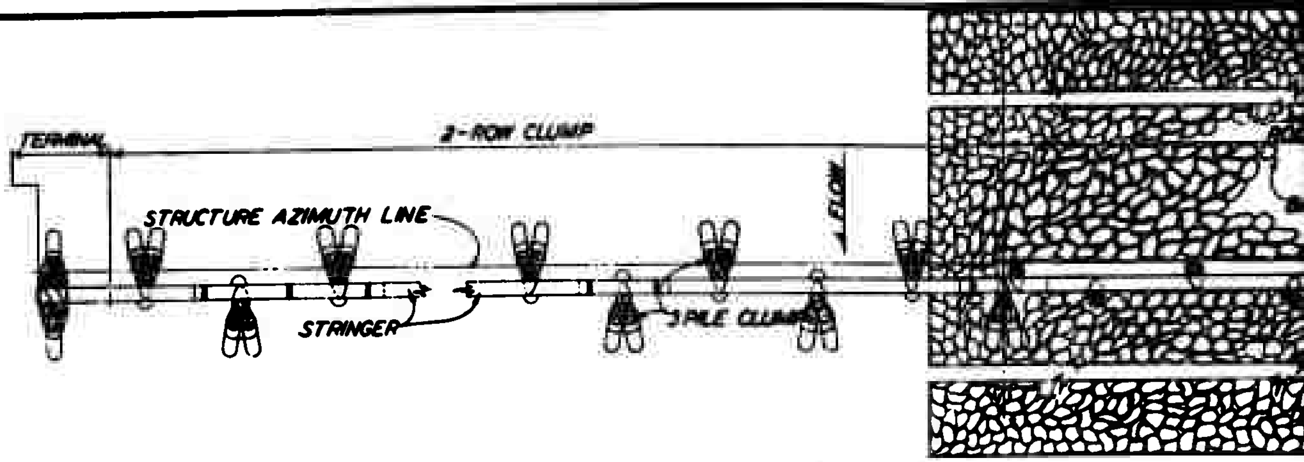
17. Quarry-run stone with fines, well graded from large to small sizes so that all the voids are filled, is the desired construction material for the trench-fill revetment. A maximum stone size of 350 lb is permitted by current specifications.

18. The standard revetment without mattress, or trench-fill revetment, can be constructed on a mass production basis with readily available mechanical equipment consisting of ordinary commercial quarry and excavating equipment: shovels, draglines, bulldozers, trucks for hauling, and sometimes wellpoint systems. This makes for speed and economy of construction.

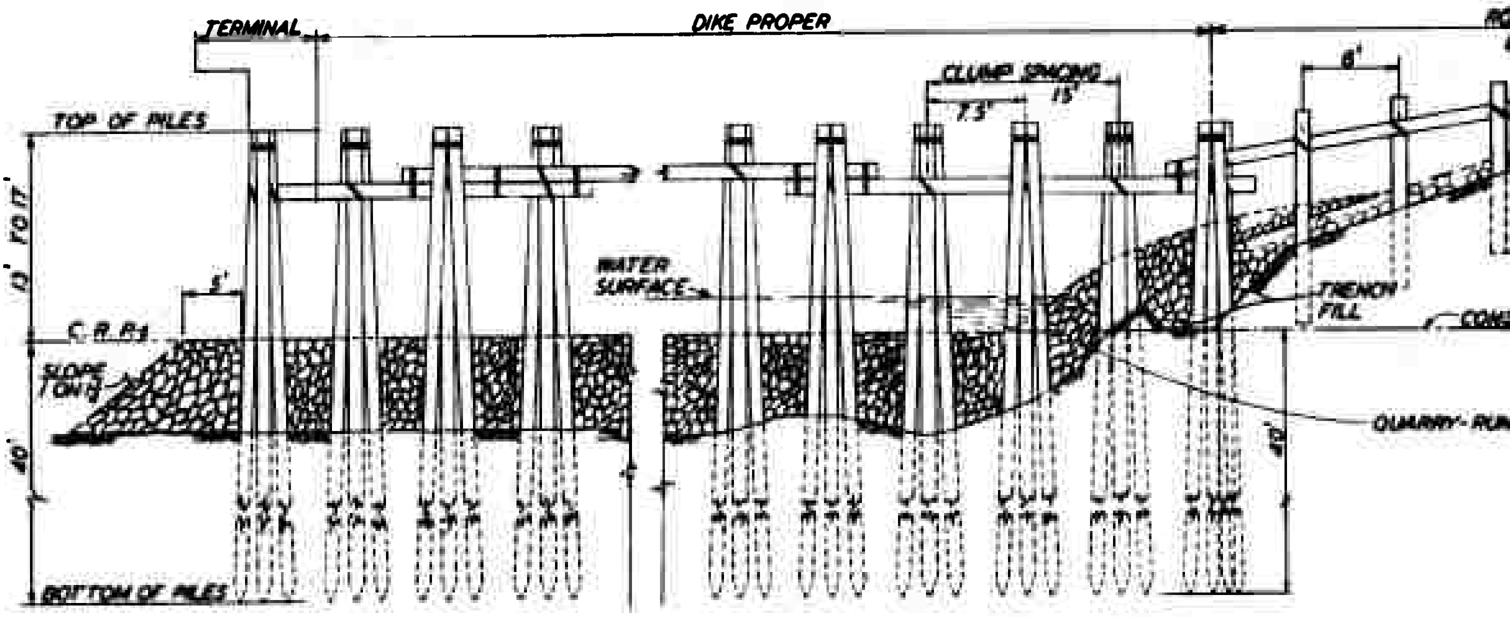
19. A typical pile dike with stone fill is shown in fig. 7. Pile revetment with stone fill is of identical construction except that a tie-in with standard revetment or with stone-fill revetment replaces the bank head. The pile structures may be either 2-row clump as shown, 3-row clump, or 2- or 3-row single pile construction, depending on the expected severity of attack on the structure. Quarry-run stone with fines, and having a 750-lb maximum size, is used for the stone fill. The stone fill serves to protect the base of the piling from scour and to add structural support to the piling as well as to aid in the creation and protection of sediment deposits. As the untreated piling will rot within 5 to 10 years, the pile structure with stone fill is considered only the first step in stage construction. The final permanent structure is obtained by adding stone fill to about stringer level along the piling after an accretion fill has formed and prior to deterioration of the piling. Usually all except the outer 100 to 150 ft of dikes becomes buried in accretion, with the result that



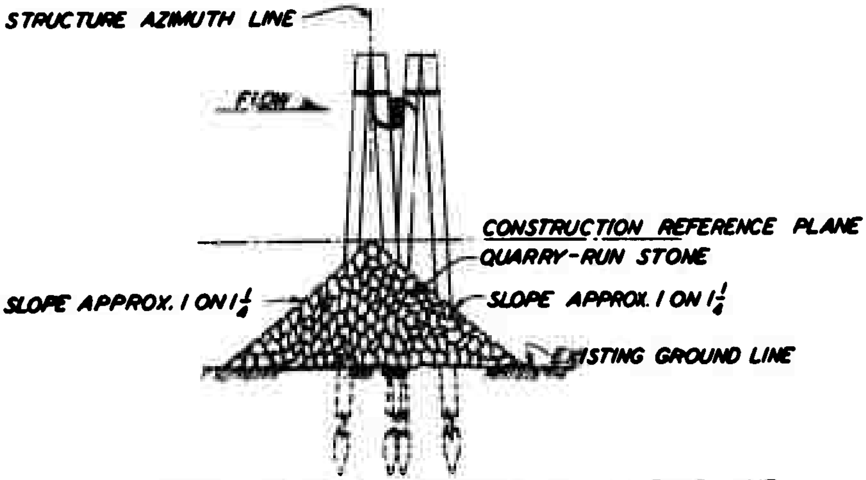
ARKANSAS RIVER BANK STABILIZATION
TYPICAL PILE DIKE WITH STONE FILL



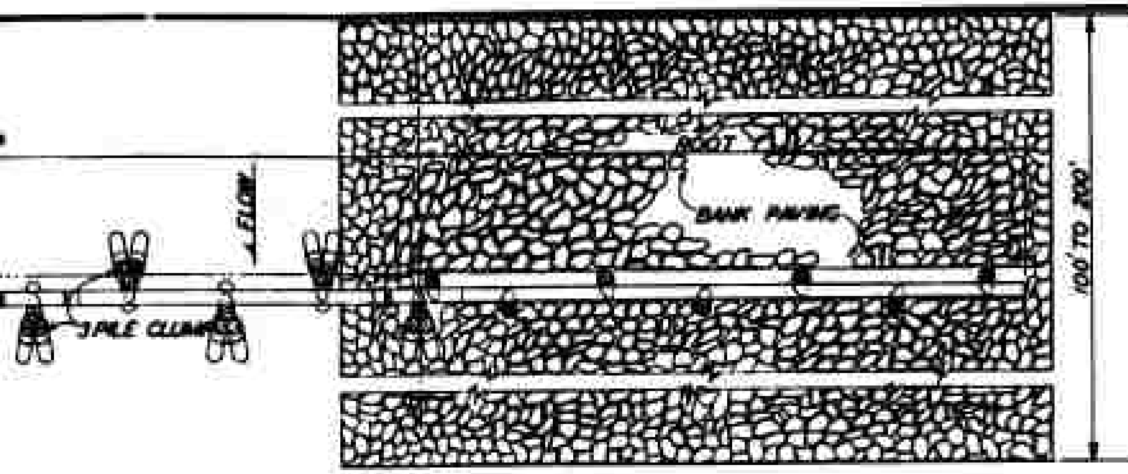
TYPICAL PLAN



TYPICAL PROFILE



TYPICAL SECTION OF STONE FILL IN DIKE LINE



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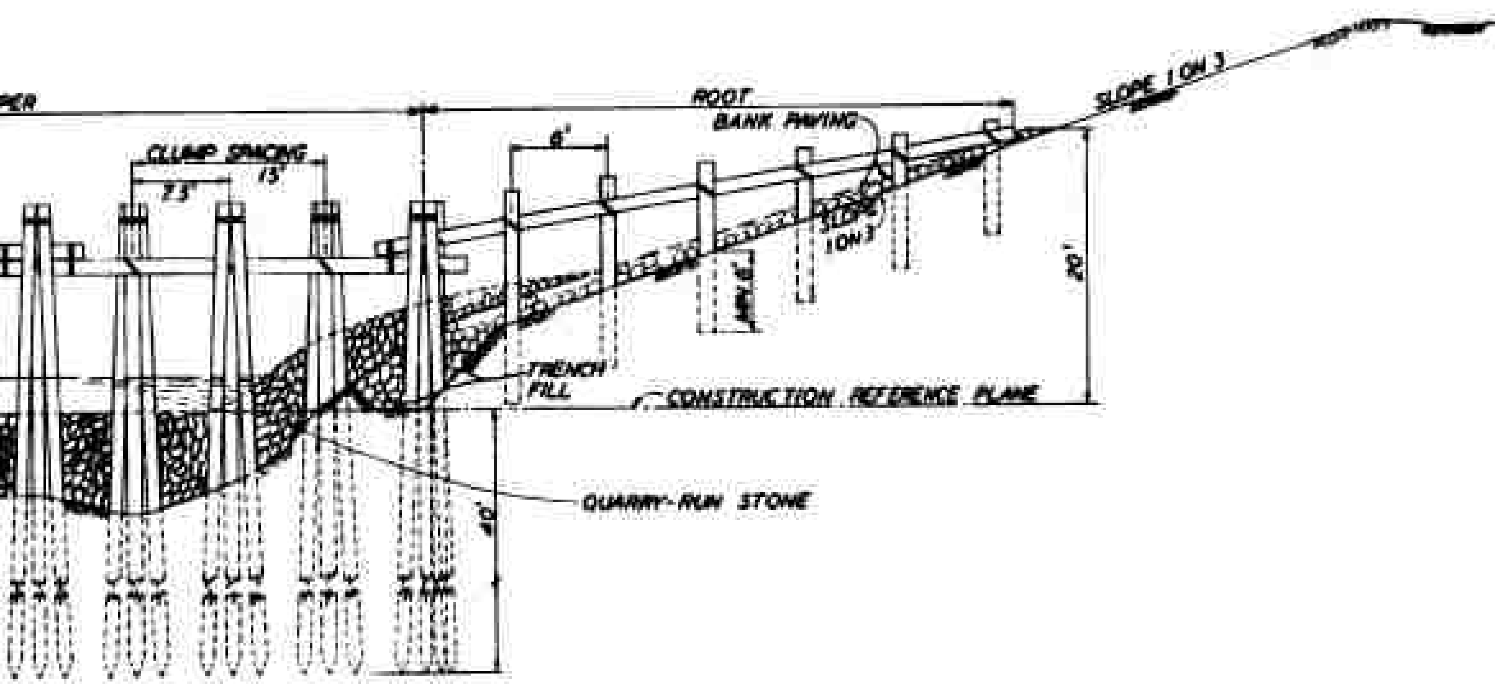

TYPICAL PROFILE

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ARKANSAS RIVER BANK STABILIZATION
TYPICAL PILE DIKE WITH STONE FILL

Fig. 7

appreciable quantities of additional stone are required only in the outer ends of the structures. In the area downstream of the vicinity of Pine Bluff, where stone is more costly, ballasted board mattresses and a relatively small amount of stone around the piling are used to protect the riverbed at the base of the piling, and longer life creosoted piling is used in the exposed outer 100-ft length of spur dikes and along the entire length of pile revetments. The tops of piling in the dikes and revetments are set at the elevation of approximately half bank-full discharge. This level has been found by experience to be adequate to control the medium and high flows. Bank-full discharges range from about 150,000 cfs in the vicinity of Fort Smith to about 275,000 cfs in the lower part of the river.

20. Typical sections of stone-fill dikes or revetments are shown in fig. 8. Quarry-run stone with fines, well graded from largest to smallest sizes so that all the voids are filled, is the desired construction

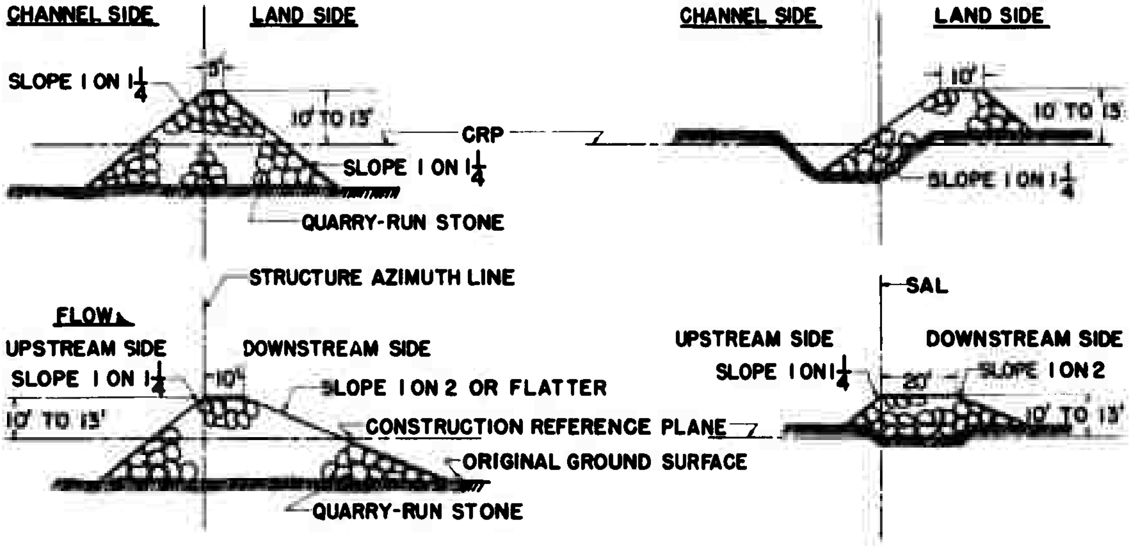

Fig. 8. Typical sections of stone-fill dikes and revetments, Arkansas River bank stabilization

material. A maximum size stone of 750 lb is permitted in the stone-fill structures. The two sections on the left side of the drawing are typical of those used where the ground line is well below CRP, and those on the right are examples of designs used where the ground line is above CRP. The crown width, side slopes, and shape of section vary depending on the strength of attack, the amount of head differential, and on whether the

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flow will be predominantly along the line of the structure, as in the case of stone-fill revetment, or normal to the structure, as in the case of a channel closure dike.

21. The decision as to whether to use a pile structure or a stone-fill structure at any particular location is governed primarily by the relative cost, which varies with the depth of the channel and with the relative availability and cost of stone and piling at the site. The proximity of bedrock to the streambed precludes the use of piling at many locations.

22. A typical steel jack jetty of the Kellner type is shown in fig. 9. The Rehfield jack jetty is similar. These types have been used to some extent on bars and on the convex side of bends in the area between Short Mountain Dam site and Fort Smith.

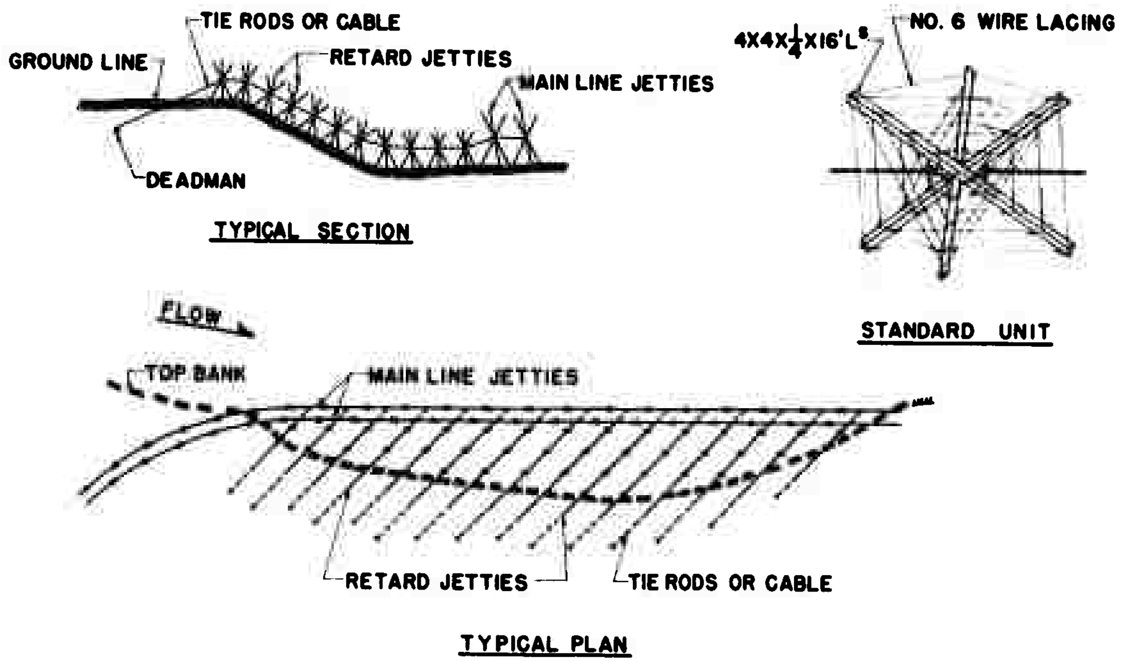

Fig. 9. Steel jetties, Arkansas River bank stabilization

23. Cutoff channels to improve alignment and achieve economy of construction and maintenance are developed by the excavation of a relatively small pilot channel which is then enlarged to full channel size by the scouring action of the riverflow. Design of the pilot channel to insure enlargement is based on the theory that the tractive force of the flow in the pilot channel must be greater than that in the abandoned river channel.

This is illustrated by the diagram and equations shown in fig. 10. A pilot channel tractive force 1-1/2 or more times the tractive force of the river is desirable for satisfactory enlargement of the pilot channel. The entrance of the pilot channel should be located in the concave bank of the bend, well upstream from the point of inflection, as illustrated, in order to avoid difficulties which may arise from excessive diversion of bed load such as occurs at locations on convex bars. A total of 15 pilot channels have been constructed since 1950 with bottom widths ranging from 20 to 130 ft. Six other pilot channels are under construction, and two more are proposed.

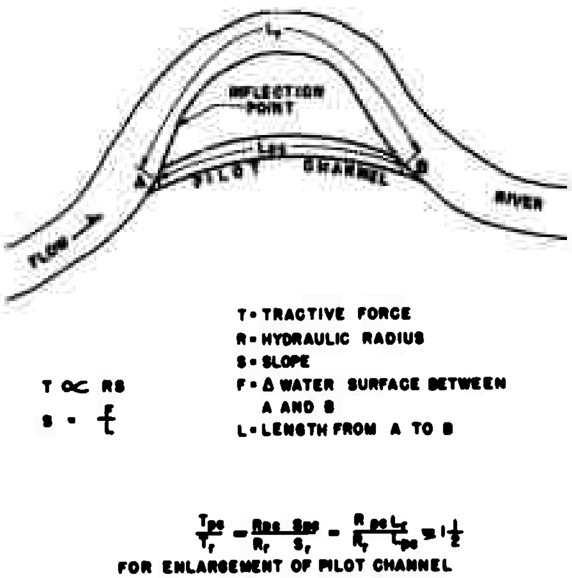

Fig. 10. Pilot channel design criteria, Arkansas River bank stabilization

24. Typical installations of some of the types of structures described previously are shown in figs. 11 through 17.

Design deficiencies and problems

25. No particular design deficiencies have been noted as long as the work is constructed in conformance with the criteria for alignment described in this paper. Extensive damages to revetments have occurred, however, at a few localities where the revetments were exposed to direct attack because of delay in completion of alignment control works upstream. Some uncertainty exists in regard to the best orientation and spacing of dikes, and improved engineering criteria and procedures for laying out dike systems are needed in place of the present methods that are based on judgment, experience, and rules of thumb. Uncertainty also exists with respect to the optimum size and shape of the toe-trench fill in the trench-fill revetments. Improved knowledge in these areas of uncertainty will require close analysis of the performance of existing structures in the prototype, and could be enhanced by programs of laboratory study in models or flumes.

Fig. 11. Standard revetment with mattress under construction

Fig. 12. Trench-fill revetment

Fig. 13. Pile dike with stone fill

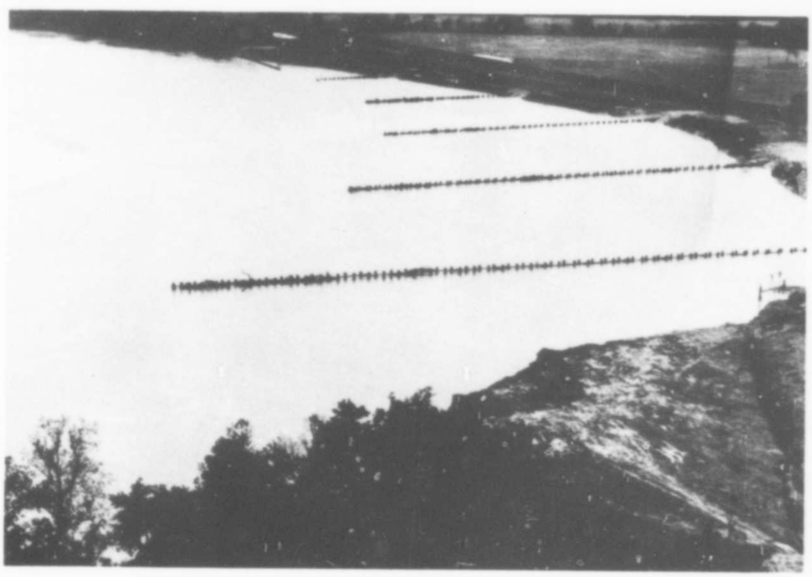

Fig. 14. Pile dike system

Fig. 15. Stone-fill dike under construction

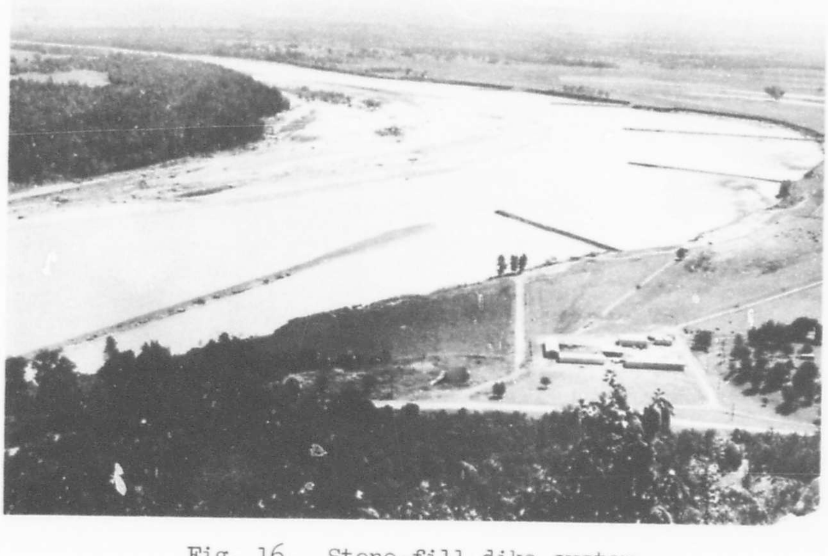

Fig. 16. Stone-fill dike system

Fig. 17. Cutoff channels

<u>Status of work</u>

26. Much of the work of stabilizing the Arkansas River has already been done. The work performed prior to Fiscal Year 1961 was limited to specific critical localities where required to protect levees, bridges, utility crossings, and other improvements that were in imminent danger of being damaged or destroyed. In Fiscal Year 1961, Congress began making available funds for carrying out the overall stabilization program for the multiple-purpose project. Expenditures in Fiscal Year 1962 amount to about $16.7 million, bringing the total expended through Fiscal Year 1962 to $55.6 million. The entire $130-million stabilization program is scheduled for completion in Fiscal Year 1968.

CHAPTER IV

CHANNEL STABILIZATION ON THE MISSOURI RIVER

by

D. C. Bondurant*

1. The Federal Government has been engaged in the improvement of the Missouri River for over 100 years, although prior to 1884 the work was restricted primarily to the removal of snags. From 1884 to 1902, the program was under the jurisdiction of the Missouri River Commission, and subsequent to 1902 has been assigned to the Corps of Engineers. A 6-ft navigation project was authorized in 1912 for the section from Kansas City to the mouth, and was extended in 1927 to include the section from Sioux City, Iowa, to Kansas City. The current 9-ft project from Sioux City to the mouth was authorized in 1945, at which time the project length was 762.5 miles. Cutoffs and various other adjustments have reduced this length to 732 miles (1960).

2. The development of channel stabilization criteria and methods has been a process of evolution by experience, leading to what we currently consider adequate criteria for our particular stream, but leaving in its path some inadequate segments which are firmly incorporated into the system. Inasmuch as the agenda for this meeting emphasizes the statement of problems, it is apropos to state that our major problem at this time is the development of a method whereby these unsatisfactory segments can be adjusted to derive acceptable results.

3. During this developmental period there has been little or no theory available in respect to channel stabilization; indeed, our theoretical competence today, other than some knowledge of sediment transport mechanics, is essentially limited to the ability to explain in technical terms why the criteria developed by experience are correct. This is a condition which must be corrected if we are to avoid another period of trial and error in a future complicated by the construction and operation of main-stem and tributary reservoirs.

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4. We are today noting indications of future problems due to reduction of main-stem transport capacity by elimination of flood peaks (complicated by upstream drouth conditions) without a corresponding reduction of sand contribution by several major tributaries. In addition, since we know that optimum meander dimensions of a meandering stream are functions of both dominant discharge and sediment load, it is reasonable to anticipate a future variation in our stabilization criteria. While these factors point up a need for technical research, they need not disturb our present operations. Knowledge of sediment transport mechanics will help to delineate any changes needed to reinstate balanced transport when necessary, and the problems due to changes in optimum meander dimensions will arise mostly in maintenance procedures. At the present time the status of various tributary reservoir proposals is in too great a state of flux to permit a strict evaluation.

5. The current criteria as developed for the stabilization of the Missouri River have generally been adopted on the assumption that the best channel segments are those requiring the least maintenance, and that these should serve as guides. It might be pertinent to state that it was fortunate that the early efforts were geared to navigation requirements, for many of the factors involved in establishing a satisfactory navigation channel have proved to be important considerations in establishing a stabilized channel. There might be some discussion as to the order of priority; however, without dwelling unduly on this item, our primary criteria are:

a. Continuity. Our experience has proven beyond doubt that a significant change in any one meander loop will be reflected in several adjacent loops, i.e. a lengthening of a major bend in a balanced reach will tend to initiate chute cutoffs in upstream bends. Since most natural meandering streams consist of a series of meanders distorted by bluff contacts or resistant plugs, they also consist of a series of unbalanced reaches; thus, in the overall planning it may be found that a loop cutoff will actually balance one reach, while a judicious lengthening of a bend will help balance another. As a matter of expediency, the planning is normally done in segments reaching from one bluff contact or other hard spot to another; however, each reach is also considered with respect to upstream and downstream reaches.

b. Alignment. A smooth alignment without bank irregularities, lunate bends, or other irregular formations is necessary.

The angle of entrance into each bend must not be too sharp, otherwise the flow might "bounce" and traverse an irregular path through several succeeding bends. There will be an optimum bend radius at which an approximately rectangular channel will follow the bend with a minimum attack on the banks. A bend which is too sharp will subtend a deep, narrow channel with excessive bank erosion. One which is too flat will subtend a wide, shallow, irregular channel with traveling bars tending to move along the concave bank. Straight reaches, unless held in place by natural formations, are avoided. One cardinal rule, however, is that any reach which has maintained itself over the years is fitted into the overall alignment with as little disturbance as possible. Fortunately, on the Missouri River, bend radii initially instituted for navigation have generally proved to be adequate with respect to channel formation and maintenance.

c. Channel width. Optimum width is the maximum which can be maintained without the formation of center bars. In a bend of optimum radius, the stream will maintain such width. In the crossings, where the flow crosses the channel between bends, the width perpendicular to the flow line should be equal to that in an optimum bend. This means, in general, that the width perpendicular to the channel banks should be approximately 85% of the bend width.

6. In speaking of optimum dimensions, there is by no means any attempt to indicate that the provision of such dimensions will eliminate the need for stabilization structures. At the absolute minimum, bank protection would be required to prevent normal meander progression, and some downward extension therefrom would be desirable to restrict the crossing width. In practice, we use extensive structures to shape the channel and others to hold it in those sections where, for one reason or another, optimum dimensions cannot be obtained; however, once a bend has been shaped to proper dimensions only the bank revetment and downstream extensions thereof remain active.

7. Shaping structures normally consist of primary lines parallel to the desired flow line, with connections to the existing bank at intervals of from 2 to 2-1/2 times the length of the next upstream connection. These parallel lines provide a smoothly aligned bank which can be given a permanent rock protection as fill accumulates behind the dike. The perpendicular dikes and trail structures previously utilized in many instances left a rough bank line in which the timber piling was subject to deterioration and ice damage.

8. Revetment structures consist of a primary line along the desired alignment of the concave bank and downstream therefrom as required to constrict the crossing. If the proposed alignment is at an appreciable distance riverward from the existing bank, the structure will first serve as a shaping structure and may require bank connections.

9. Many types of shaping and revetment structures have been utilized during the development period; however, we now use only rock-filled pile dike, rock dike, and rock revetment unless some unusual circumstance dictates a deviation therefrom.

10. Pile dikes are the usual 2- or 3-row pile clump or 1- or 2-row single pile connected by horizontal stringers, and need no further description. We have, however, almost wholly abandoned the use of a foundation mattress; in fact, in many instances the pile dike is used only as a guide for rock placement. In all cases, the dike is filled with rock. Where the dike forms the bank alignment, the base rock is placed immediately after the driving of the piling, and the rock is subsequently brought to grade as the fill accumulates. In connecting dikes the rock fill may be brought to grade as soon as driving of the piling is completed.

11. Rock dikes without guide piling are placed where flow velocities are not excessive and the depth below grade is less than about 7 or 8 ft, the selection being strictly an economic balance between the cost of the piling and the cost of the additional rock required for placement without the piling. These dikes are usually brought to grade in one step. Conditions for selection of the rock dikes normally restrict their use to connecting dikes or chute closures.

12. Rock revetment is utilized where the alignment is on or landward of the existing bank. The toe-trench type has been adopted as standard except that in instances where a sharp angle of attack is unavoidable a rock revetment with mattress extension may be used.

13. The toe-trench revetment has proved to be particularly suitable for the Missouri River, where it is seldom necessary to place the toe more than 9 or 10 ft below the normal low-flow elevation. The use of a well-graded rock so that all voids are filled insures against leaching of the underlying soil, yet the rock will settle and be essentially self-healing in the event damage does occur. The excess toe rock will settle to stop

undercutting; however, it has been noted that the rough surface of the revetment causes the high-velocity flow to stand out from the revetment so that undercutting is inhibited. This latter factor might not be notable in streams of depth such that the high-velocity flow would still be on or immediately adjacent to the revetment even with the standout, but it is sufficiently effective on the Missouri that at some locations depths of 30 ft below the toe elevation obtain without damage to the revetment. The primary testimony on the efficiency of this type of revetment on the Missouri is the fact that there has not been a failure of consequence in approximately 20 years of use.

14. A major factor in the planning for the stabilization of the Missouri is that the river is small enough to be moved into a desirable alignment and large enough that damages accruing from the uncontrolled stream provide a rational economic evaluation for its control. Thus, we are not confined to an existing channel per se, but have reasonable latitude in alignment planning. In earlier years the channel was moved by constructing dikes to contract the channel on one side and cause erosion of the banks on the opposing side until the desired position was attained. Development of the toe-trench revetment, however, has permitted excavating a trench along the final alignment of the concave bank, placing the revetment in the dry, and opening a pilot channel by which the river is persuaded to move itself into position. The persuasion, of course, requires supplementary closure or contraction structures; however, the newer method provides decided economies in both funds and time.

15. It was earlier mentioned that development by experience had left in its path several unsatisfactory segments of channel. These are almost universally bends which are either too flat or too sharp, resulting usually from an early lack of appreciation of the value of an optimum bend radius and consequent reluctance to take the land required for a proper location. In the case of the overly sharp bend, the effective channel width is less than is desirable, and bank erosion tendencies are stronger; however, there is little to do except to accept a somewhat higher maintenance cost and take additional pains to lead the flow properly through the crossing into the next bend. In a flat bend, the channel is wide and shallow with intermittent center bars and a tendency for the flow to shift from place to

place. Here, a search must be made for a method to hold the flow in a reasonable channel against the concave bank.

16. The major difficulty lies in the fact that the channel must carry high as well as low flows; thus, it cannot be simply contracted to low-flow dimensions. The obvious solution is to maintain a low-elevation convex bar which will be submerged during a sufficient portion of the time to prevent the growth of vegetation which would, in turn, encourage further deposition until the high-water channel was lost. We are experimenting with rock dikes constructed only to the elevation to which bar formation is desired. The outlook to date is promising, although some operators claim that flow velocities are increased too much.

17. There are locations where spurs have been constructed, mistakenly, along concave alignments, usually in flat bends. These seriously increase the reach roughness, develop scalloped bank lines, and engender irregular flow alignments in some cases. Tests of L-head additions, i.e. downstream extensions along the desired alignment, have given excellent results. To date the optimum elevation or length required for these extensions has not been determined; however, it appears that construction to low-water elevation and to a length one-half to two-thirds the distance to the next structure might be adequate. One difficulty in constructing these elements to an elevation less than that of the spur to which they are attached is that scour occurs landward of the junction when they are overtopped. This may require use of a permeable, rather than an impermeable, structure until the fill is established and vegetated.

18. The 1962 navigation season has emphasized that a very serious problem can occur with a sediment imbalance resulting from a hydrologic cycle variable both in time and space. We have long anticipated that the regulated flows of the Missouri might have difficulty moving tributary sediments formerly flushed out by floodflows, but have not investigated very deeply because of uncertainty as to future tributary reservoirs.

19. For several years prior to 1962, above average discharge from the major sediment-contributing tributaries, the Platte and Kansas Rivers, has occurred together with subnormal runoff in the upper basin. During the first six months of the 1962 season, runoff in the reach beginning almost immediately below the Gavins Point Dam was well above normal, and flows

averaged 10 to 20 percent above navigation requirements. This was sufficient to initiate movement of an appreciable portion of the previously deposited tributary fans, and reduction of the flow to approximately navigation requirements during the final two months left several reaches in an overloaded condition. Random bars formed and caused a number of groundings in areas where few, if any, groundings had occurred in prior years. While there were undoubtedly other contributing factors, the downstream time progression pointed a definite finger at a sediment imbalance.

20. We are now at a crossroads in the Missouri River navigation project. There can be no question but that the long experience in Missouri River development has engendered layout and construction criteria well adapted to the stream. Nearing the end of our construction schedule, however, we are finding some areas in which our experience is inadequate, or which would require too long a period for prototype testing, for the final wrap-up. We are initiating a more scientific study of these areas, particularly with respect to the anticipated variation in future sand load.

CHAPTER V
CHANNEL STABILIZATION PRACTICES ON MIDDLE RIO GRANDE IN NEW MEXICO

by

C. E. Hyvarinen*

1. The Middle Rio Grande Valley lies in central New Mexico, extending about 183 river miles from the Cochiti Indian Pueblo, about 48 river miles upstream from Albuquerque, to the upper limits of Elephant Butte Reservoir. The river is generally confined by the Rio Grande Floodway through most of this reach. This floodway, consisting of levees and channel rectification works for flood protection of agricultural and urban properties, was built by the Middle Rio Grande Conservancy District in the early 1930's. Rehabilitation and improvement of the floodway was authorized in the Flood Control Acts of 1948 and 1950 as a joint undertaking by the Corps of Engineers and the Bureau of Reclamation. Construction on the floodway is now under way and will provide a stabilized channel. Fig. 1 is a map of Rio Grande.

2. On the basis of the drainage pattern and the scope of the development along the river, the floodway was divided into three units. The Albuquerque Unit, completed a few years ago, was treated separately, because the more extensive development required special consideration for flood control. The Cochiti to Rio Puerco Unit is the longest, extending about 105 river miles from Cochiti to the mouth of Rio Puerco. Construction work is scheduled for completion in September 1962. This river reach includes the 20-mile Albuquerque Unit. The Rio Puerco to Elephant Butte Unit extends downstream from the confluence of Rio Puerco to the upper end of Elephant Butte Reservoir. Construction is not scheduled at this time. Since the Cochiti-Rio Puerco Unit is practically completed, and is the longest and most important reach, this paper will generally be limited to stabilization works in this section of the river.

3. On the Middle Rio Grande riverflows are of two types generally.

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* U. S. Army Engineer District, Albuquerque, New Mexico.

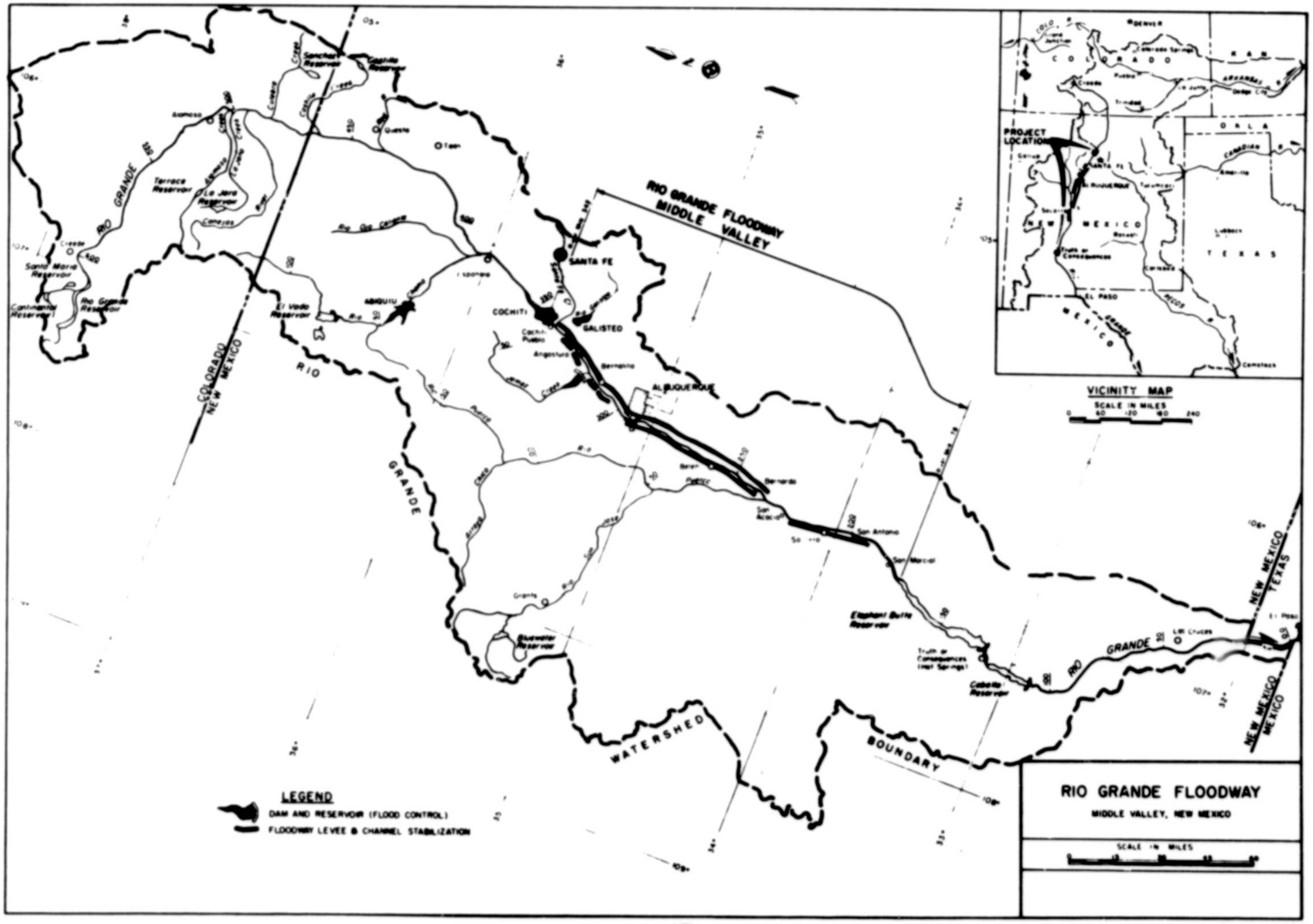

Fig. 1. Map of Rio Grande

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One type, spring flows (April through June), results from snowmelt and is often augmented by general rainfall in the basin. The other type, flash flows, occurs during the summer months, extending occasionally through October. The spring flows are characterized by a gradual rise to a comparatively moderate discharge that is usually maintained for about two months with peak flow of shorter duration. Volume of runoff is large. The flash flows rise sharply to a peak and recede rapidly. While the volume of runoff is small, the peak flows are comparatively large. Flows above 5000 cfs are considered floodflows. Based on available records at Bernalillo, just upstream of Albuquerque, the maximum flows are about 25,000 cfs for spring flows, and 27,000 cfs for summer flows. Historic floods undoubtedly have been much greater. Estimates of flows, based on studies of historical accounts of spring floods that occurred in the 19th century, range as high as 125,000 cfs.

4. The slope of the river is 4 to 5 ft per mile. As previously mentioned, the river is generally confined by the leveed floodway. The floodway width for the most part is about 1500 ft. The Rio Grande occupies a channel about 800 ft in width between the floodway levees. Between the channel and the levee are growths of willow, cottonwood, and salt cedars of varying density. For all practical purposes, the channel has no banks and the average river level is at or above the level of the area behind the levees.

5. Rio Grande carries a large suspended-sediment load. At Cochiti, it averages about 0.194 percent by weight, and at Bernardo, just upstream from the mouth of Rio Puerco, it averages 0.454 percent. The percentage of sand increases rapidly with increase in flow. More sediment is entering the floodway than is removed, and as a result aggradation averages about 3 ft every 50 years.

6. The sinuosity of Middle Rio Grande may be classified as mild to moderate. There are some relatively straight stretches of channel. The channel is braided, probably because of the steep slope and high sediment concentration. The levees, completed in 1936, were constructed in general conformity with the river channel meander pattern. Sandbars and growth of vegetation within the leveed areas have an adverse effect on stabilization of the channel. During spring floods particularly, the channel scours deep

and shifts toward the levees. The channel may scour to depths up to 20 ft. Numerous failures of these levees by undermining and slumping have been experienced. In 1941 when flows reached 25,400 cfs, there were 25 levee breaks and a total of 80 locations where emergency floodfighting was required. Again in 1942, flows reached 24,200 cfs, resulting in two levees failures and requiring floodfighting at 50 locations. Therefore, stabilization of the channel is necessary to prevent levee failures. Furthermore, there is a pressing need for water salvage by reduction of channel area exposed to evaporation, better drainage, and sediment-carrying capacity which a stabilized channel will promote.

7. Stabilization works for the Middle Rio Grande, with its easily erodible streambed, necessitated development of a flexible system which would function under the severe channel scour encountered. Furthermore, establishment of a protective bank along the levees was required. To provide control of the large magnitude floodflows and sediment, it was found that dams would also be necessary along with channel stabilization works. Locations of the dams are shown on the basin map. Jemez Dam was completed in 1953, Abiquiu Dam is under construction and is scheduled for completion in 1962, and Cochiti and Galisteo Dams are in the preconstruction planning stage. The floodway is designed for discharges of 20,000 cfs except in the Albuquerque Unit where the design is for 42,000 cfs. After completion of the dams the floodway stabilization works will: (a) maintain the improved channel, and protect the levees against reservoir releases and infrequent floods larger than those of record which cannot be completely controlled by the dams, and (b) effect water salvage and major drainage through channel degradation.

8. When the floodway was built, some stabilization works were installed by the Middle Rio Grande Conservancy District. The Corps of Engineers under emergency flood-control authorities also installed levee protective works at critical locations. These installations consisted of single- and double-row pile jetties. These types of jetties are no longer utilized. While they provided partial protection, experience showed that their effectiveness in controlling and arresting scour, under even moderate flows, was inadequate.

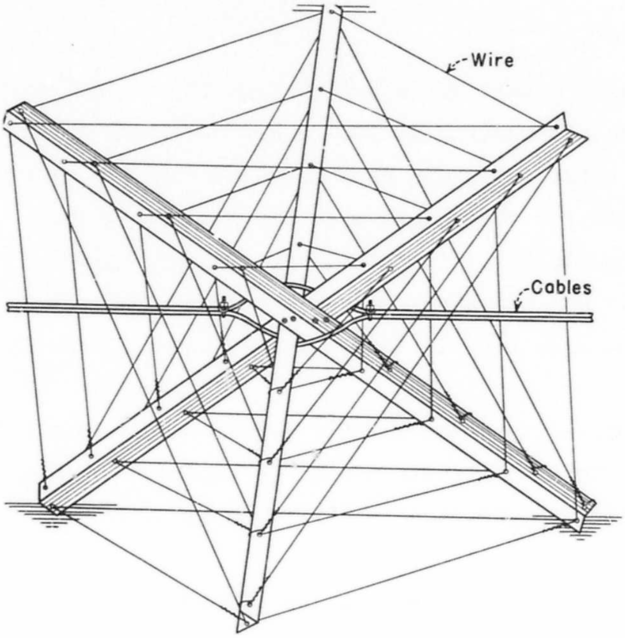
9. It was apparent that a more effective means of providing

protective works was needed. Use of the Kellner jetty system was undertaken in 1950 by the Albuquerque District on emergency flood-control work. The Kellner jetty system had been used by the Santa Fe Railway in maintenance of way along the Rio Grande and Arkansas Rivers for many years, and with such success that it has largely superseded other types in use by the railroad. The Kansas State Highway Department has also successfully utilized this system for protection of highways and bridges from bank cutting. The system was developed by the Kellner Jetties Company, Topeka, Kansas, in the early 1920's. Mr. H. F. Kellner started his experiments on a small stream near Topeka utilizing a unit of three willow poles tied together and laced with wire. Experimentation with a variety of shapes and materials led to the present structural steel angle unit. The system was formerly covered by patents which have expired.

10. The Kellner jetty system is well adapted to siltladen streams which are subject to considerable channel scour. The system is permeable, flexible, and readily conforms to channel scour. The lines of jetties retard the velocity of the streamflow because of the obstruction offered by the components of the jetty units. This reduction in velocity allows deposition of some of the suspended sediment, and eventual formation of a stabilized bank within which rapid vegetal growth further strengthens the area, and also reduces the channel area exposed to evaporation. This promotes water salvage, an important consideration in semiarid regions.

11. The individual unit of the system is called a "jack." It consists of three 16-ft-long, 4- by 4- by 1/4-in. structural steel angles bolted together at their centers forming planes which are laced with wire. A lighter unit of 3- by 3- by 1/4-in. angles is also used. Figs. 2 and 3 illustrate the standard jack.

12. The jacks, spaced at 12-1/2-ft centers, are connected by two strings of wire rope to form jetty lines which in turn are connected together in a pattern or jetty field. Two types of lines are employed, diversion lines and retard lines. The diversion lines, usually two in number, are placed roughly parallel to the levee or bank to be protected, and along the desired location of the channel. The retard lines extend from the diversion lines to the levee where the lines are anchored. These retard lines are placed on an angle of 45 to 67-1/2 deg to the diversion



NOTES:

1. UNIT IS USUALLY 16 FT BY 4- × 4- × 1-1/4-IN. ANGLE IRON LACED WITH NO. 6 WIRE
2. CABLES ARE USUALLY 3/4-IN. DIAMETER OR LARGER
3. UNIT SHOWN IS PLACED ON 12-1/2-FT CENTERS

Fig. 2. Sketch of Kellner jetty unit

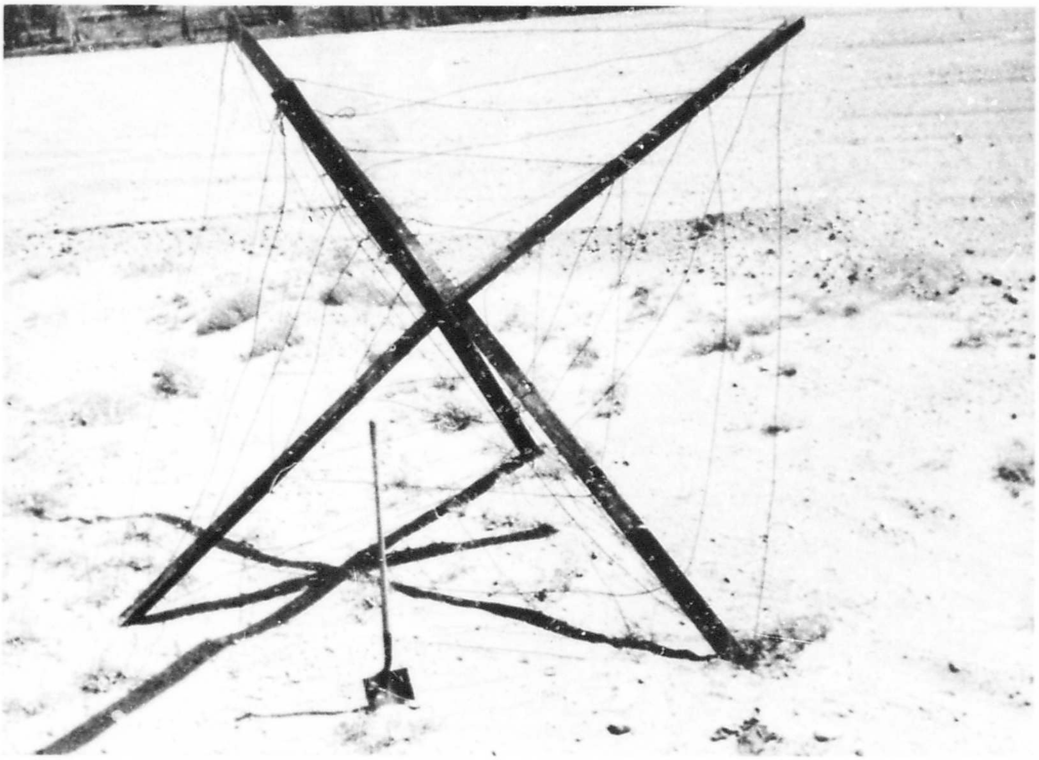


Fig. 3. Jetty unit assembled

line. At major bends spacing of retard lines varies from 125 to 250 ft. Single diversion lines with retard spacing at 500 ft are utilized at the less critical areas; however, wire ropes are used for anchorage between the retards. Anchorage is provided for the retard lines by deadmen or by anchorage to trees. In the double diversion line jetty fields, 3/4-in. wire rope is used throughout. For the single diversion lines, 9/16-in. wire rope is used, and 1/2-in. wire rope is used for the lighter unit retard lines.

13. Fig. 4 shows a typical layout of a jetty system on Rio Grande for the Cochiti to Rio Puerco Unit. Adequate design is important. Other rivers with different conditions may require jetty fields of different density.

14. General principles usually followed in the layout of a jetty

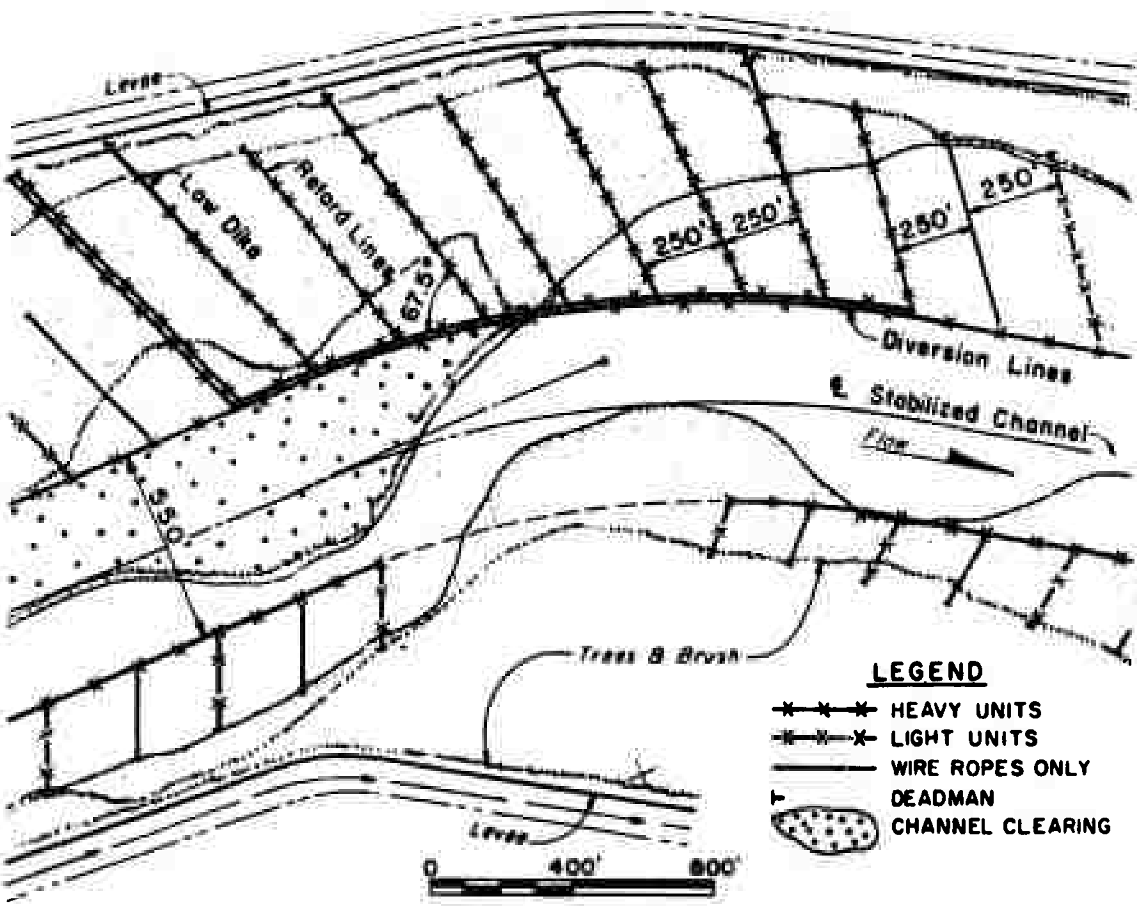


Fig. 4. Typical layout of jetty system on Rio Grande Floodway, Cochiti to Rio Puerco Unit

field are contained in the following publications:

(1) Civil Works Investigation, Report on Measures for Bank Protection, Use of Kellner Jetties on Alluvial Streams, 1953, Albuquerque District.

(2) Use of Steel Jetties for Bank Protection and Stabilization in Rivers, 1956, Carlson and Enger, Bureau of Reclamation.

(3) Hydraulic Model and Prototype Studies of Casa Colorado Channelization, Middle Rio Grande Project, New Mexico, Hydraulic Laboratory Report No. Hyd-477, 1961, Bureau of Reclamation.

(4) "Stabilization of the Middle Rio Grande in New Mexico," 1961, R. C. Woodson, Chief, Engineering Division, Albuquerque District, ASCE Proceedings.

15. The Civil Works Investigation report summarizes the experience of the Albuquerque District, and covers some jetty installations of other agencies at small isolated locations prior to 1953. The paper by Carlson and Enger, and the Hydraulic Laboratory Report present information on the hydraulic model studies and prototype investigations on the Casa Colorado experimental area, which is located near the lower end of the Cochiti to Rio Puerco Unit. Design curves and nomographs developed are presented which can be used with judgment in the layout of a jetty field. This experimental work was very valuable in determining the basic layout for the jetty fields. The paper by Woodson describes the stabilization program on the Middle Rio Grande, and much of the material has been used by the author.

16. The following tabulation of comparisons of streamflow velocities in the channel and jetty field at the Casa Colorado installations shows the great reduction in velocities (3.0-3.6 to 1) which induces sediment deposition in the jetty field.

Comparison of Velocities in Channel and Jetty Field

Casa Colorado Prototype Jetty Installation*

Location	June 1957			May 1958		
	Q, cfs	V_{max}, ft/sec	Depth, ft	Q, cfs	V_{max}, ft/sec	Depth, ft
West jetty field						
Channel	3600	6.9	4.4	8020	9.7	7.0
Jetty field	250	1.9	1.6	720	2.9	3.9
Total	3850			3740		
East jetty field						
Channel	3220	8.0	7.6	8440	8.2	4.0
Jetty field	780	2.2	2.3	460	2.7	2.8
Total	4000			8900		

* Constructed in 1956.

17. Construction of a small jetty field is very simple. A work crew of 12 common laborers and a supervisor can assemble and place up to 25 jacks per day. The jacks are erected with hand tools at the site and carried to position in the jetty field by 4 or 5 men. If the work is being done in water, they are rafted into place. Usually about 16 units are assembled at a time. The cables are threaded through units and clamped. Natural vegetation is disturbed as little as possible.

18. Where larger fields are used, as on the Rio Grande, mobile, mechanized assembly and placement equipment is being used. This equipment consists of a tractor, mounted with a light crane drawing a flatbed trailer. The front part of the trailer contains racks for angles and wire rope. Behind this is the assembly jig for the jacks. The crane handles the materials from the delivery trucks. To start the line, angles are placed in the jig and bolted with pneumatic wrenches. The wire rope is passed around the angles and clamped. The trailer moves forward about 12.5 ft placing the jack in the first lacing position where it is partially laced. The trailer moves forward another 12.5 ft which places the jack in the last lacing position where lacing is completed. Meanwhile, 2 more jacks have entered the assembly line. Thus, with each forward movement of 12.5 ft, a completed jack drops off the end of the trailer in proper position.

19. This equipment, shown in fig. 5, was devised by Miller and Smith, Albuquerque contractor. Operation of the equipment requires a supervisor and a crew of 16 men. Placement of about 125 jacks per day is accomplished under average conditions. The equipment can operate in water up to about 8 in. deep.

20. All jetty installations have been constructed by contract. Jetties are usually bid on a unit basis complete in place which includes wire rope, clamps, anchors, etc. The last contract unit price (November 1961) was $41.56 for the heavy units and $35.00 for the light units. The total cost of the Cochiti to Rio Puerco Unit of the floodway (excluding the Albuquerque Unit) is about $4,400,000 or about $50,000 per mile. About $220,000 of the total cost will be expended in channel clearing and necessary pilot channels through bars in the river. It must be borne in mind that these costs are, for the most part, for both sides of the channel.

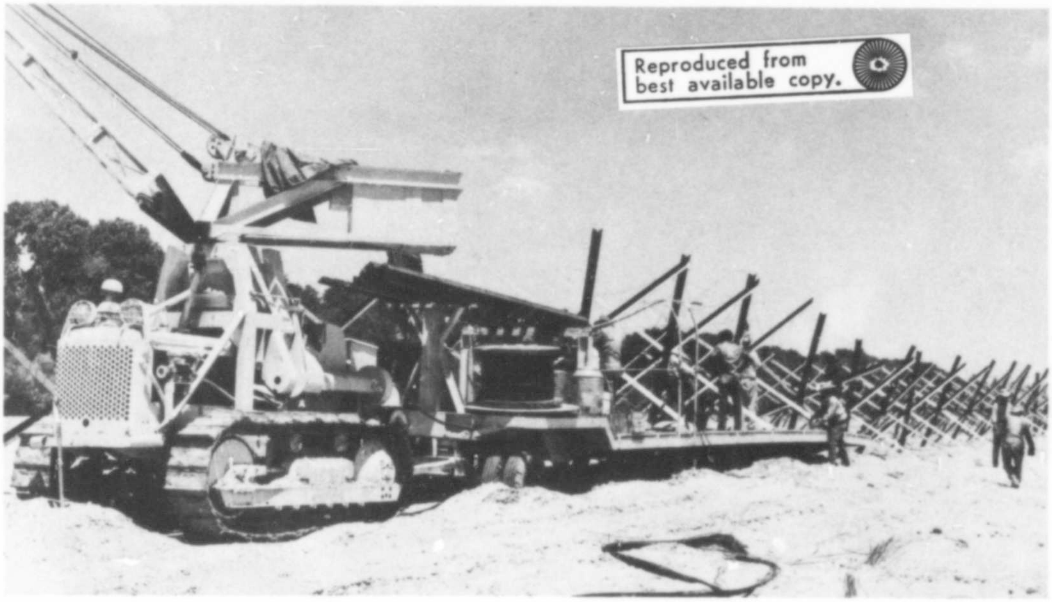


Fig. 5. Jetty assembly and placement equipment

The project is scheduled for completion in September 1962. Fig. 6 shows an aerial view of completed jetty system including a pilot channel.

21. The earliest uses of the Kellner jetty system in the Rio Grande Middle Valley were by the Santa Fe Railroad for protection of railway embankments, beginning in 1936. This work was on a limited scale with satisfactory results. The first large-scale use of the system by the Corps of Engineers was in 1953 under emergency flood-control authority. Seven jetty fields were constructed at critical locations where the levees were exposed to attack by the river. About 5600 jacks were installed. The design of the fields was similar to that shown in fig. 4 except that the retard lines were spaced at 125 ft and were considerably shorter. Fig. 7 illustrates conditions at the Bernalillo installation in December 1952 prior to construction. Fig. 8 was taken in August 1953 at the same location after installation of Kellner jetty field in June 1953. Figs. 9 and 10 show this installation in September 1953 and April 1958, respectively. Note the growth of vegetation in the jetty field.

22. A cooperative photogrammetric survey program of the Corps and the Bureau of Reclamation is now under way in the Middle Rio Grande to ascertain the overall effects of the authorized reservoir projects and

Fig. 6. Aerial view of completed jetty system including a pilot channel

Fig. 7. Emergency flood-control work, Rio Grande near Bernalillo, New Mexico, prior to installation of jetty system

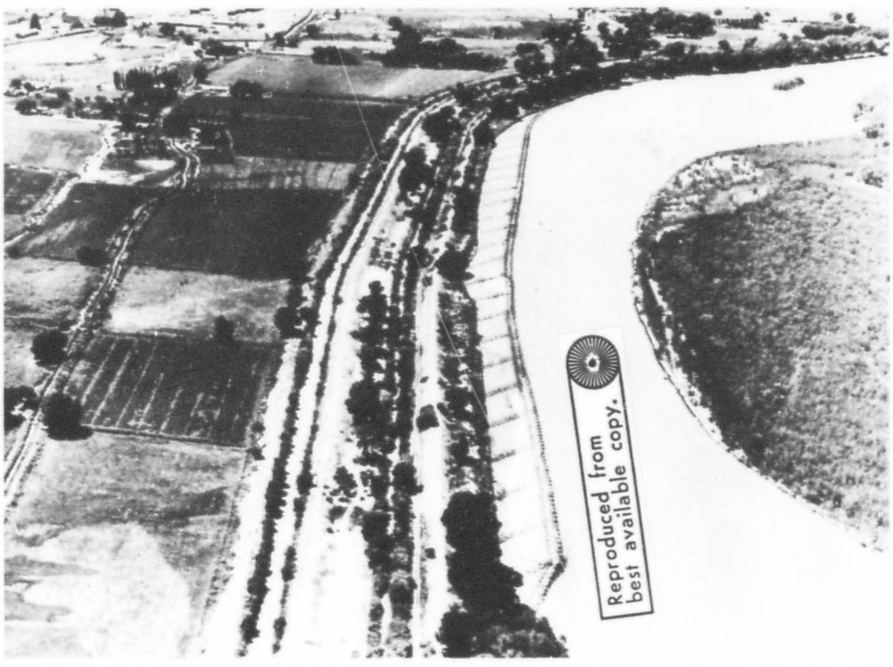


Fig. 8. Emergency flood-control work, Rio Grande near Bernalillo, New Mexico, in August 1953 after jetty installation in June 1953

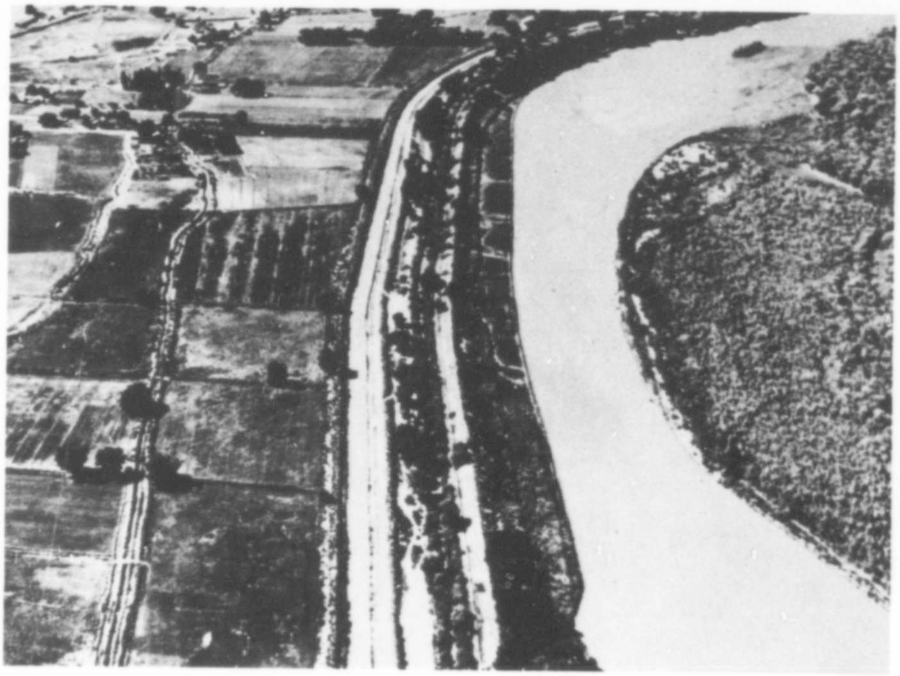


Fig. 9. Emergency flood-control work, Rio Grande near Bernalillo, New Mexico, in September 1955 two years after installation

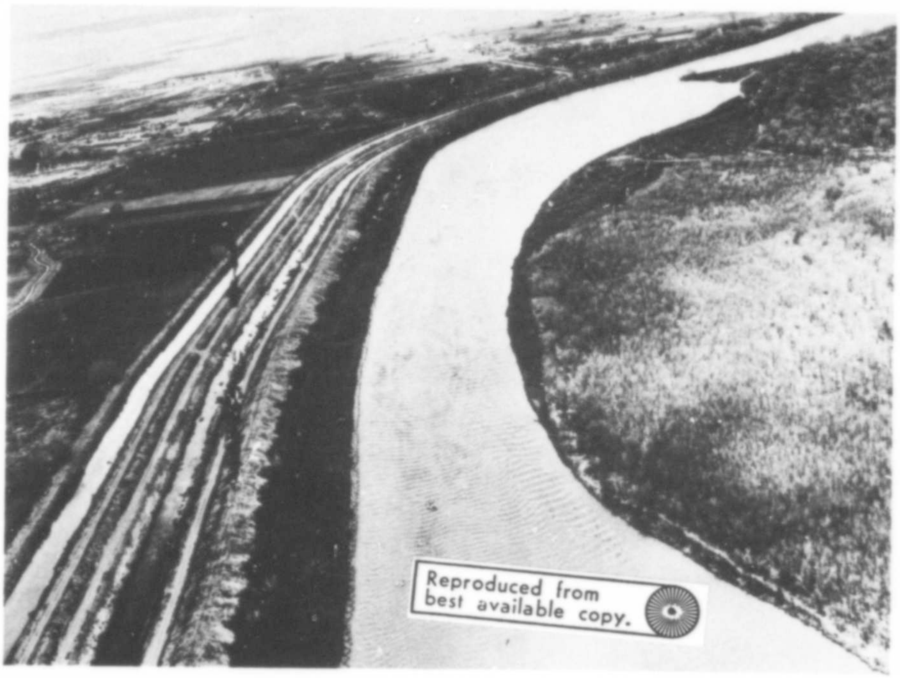


Fig. 10. Emergency flood-control work, Rio Grande near Bernalillo, New Mexico, in April 1958 five years after installation

floodway works. The initial survey contract with Abrams Aerial Survey Corporation, Lansing, Michigan, is scheduled for completion in January 1963. It is planned to conduct resurveys at about 5-year intervals or when Rio Grande flows may indicate. In addition to aerial mosaics, channel cross sections will be obtained by stereophotogrammetric methods for adaption to electronic computer determination of streambed degradation and aggradation.

23. Construction of the stabilized channel in the Cochiti to Rio Puerco reach of the floodway began in 1954 in the Albuquerque Unit. When this work was completed in 1956 about 17,000 additional units had been installed. From 1956 to 1958 no work was accomplished. Work was started again in 1959 and about 83,000 more units have been installed to date (April 1962). When the work is completed in September 1962 a total of approximately 115,000 units will be installed.

24. The 1953 jetty fields have been exposed to the following maximum peak flows:

1953	9,100 cfs, summer flow
1954	4,250 cfs, summer flow
1955	13,300 cfs, summer flow
1956	7,300 cfs, summer flow
1957	7,140 cfs, spring flow
1957	13,900 cfs, summer flow
1958	11,800 cfs, spring flow
1959	2,500 cfs, summer flow
1960	4,800 cfs, spring flow
1961	4,750 cfs, spring flow
1962	7,000 cfs, spring flow

This period is considered to be a period of subnormal flow, and conditions favorable to stabilization of the jetty fields have prevailed. The floodflows have been moderate. Sedimentation and growth of vegetation within the jetty field, which usually occurs within 2 to 3 years, have been adequate. No failures have been experienced.

25. Repair and maintenance of the jetty fields are usually accomplished during low-flow periods by installation of additional lines of jacks in areas which have been subjected to extreme scour conditions. Jetties have been effectively used in floodfighting, to prevent loss of sections of levee or river exposed to channel scour. In such a case, lines of jacks are assembled on top of the levee or bank. When the assembly is

complete with suitable slack anchor lines, it is tipped over into the scoured area. A second line may be necessary where scour is severe.

26. Maintenance costs on properly designed jetty fields are moderate. No maintenance has been required on the 5600 units installed in 1953 nor on the additional 17,000 units installed prior to 1956. As the project is still under construction, it has been necessary to estimate future maintenance costs. It is estimated that the entire 105 miles of channel stabilization can be maintained over a period of 50 years at an annual cost of $130,000, including channel clearing and replacement of 50 percent of the jetty units. This amounts to about $1250 per mile per year. The estimate is predicated on the assumption that all authorized flood-control dams will be in operation within a reasonable period after completion of the channel stabilization.

27. The effective life of a Kellner jetty field should be at least 25 years under conditions prevailing in semiarid regions, such as the Rio Grande Valley. In more humid areas, or where corrosive water is found, the effective life would be shorter. The 1953 installations by the Corps of Engineers are intact after 9 years of service with no visible signs of deterioration. The works constructed by the Santa Fe Railroad in 1936 are fully effective after 26 years of service, although additional jacks were installed in 1952. It is probable that with this replacement, the life would be extended about 25 years. No serious deficiencies in the use of the Kellner jetty system in channelization of the Middle Rio Grande are evident. It is recommended that large-scale use of the Kellner jetty system for channel stabilization on other rivers should be preceded by construction of a test reach and, if possible, investigation in the laboratory to predict performance at range of flows expected.

28. In conclusion, channel stabilization of the Middle Rio Grande is being accomplished effectively and economically by use of the Kellner jetty system. This system is particularly adapted to wide, shallow rivers of the Southwest, such as the Rio Grande, in which sediment content is high and the riverbed is subject to considerable scour during floods.

CHAPTER VI

CHANNEL STABILIZATION, COLUMBIA RIVER

by

M. C. Bubenik*

1. The Columbia River, with a drainage area slightly over 259,000 sq miles, of which 220,000 lie in the United States, is the largest stream on the Pacific Coast. The largest tributary is the Snake River with a drainage area of 109,000 sq miles. Other important tributaries include Kootenai, Spokane, Clark Fork, Okanogan, Yakima, Umatilla, John Day, Deschutes, Willamette, Lewis, and Cowlitz.

2. Climate of the basin varies between mild coastal conditions at the mouth, where annual rainfall is as high as 150 in., to the extreme temperatures in the higher mountains where precipitation is largely snow.

3. Columbia River flood or freshet stage occurs annually during the months of May, June, and July, and high stage lasts from 30 to 60 days. Maximum discharge (estimated) was 1,240,000 cfs. Mean floodflow is about 600,000 cfs at The Dalles and occurs during spring runoff. Existing upstream storage permits regulation of low flow to 75,000 cfs.

4. Tidal effects upon river stages at Vancouver, river mile 106.5, can be as much as 3.5 ft with normal variation during low water of about 2.5 ft. During periods of flood, tidal effects at Vancouver disappear. The following tabulation shows extreme tidal range at Astoria, river mile 14.

	ft
Highest tide (estimated)	12.0
Mean higher high water	7.9
Mean high water	7.2
Mean low water	1.1
Mean lower low water	0.0
Lowest tide (estimated)	-3.0

5. Channel materials are essentially medium and fine sand, and the

* U. S. Army Engineer Division, North Pacific, Portland, Oregon.

river is characterized by numerous sandbars, islands, and channels of varying width and depth.

Prevailing channel dimensions

6. There are 26 bars in Columbia River below the mouth of Willamette River that require regular maintenance dredging. In the reaches between the bars the river maintains itself, with depths ranging from 35 to 150 ft over widths of 500 to 2000 ft. Channels through the bars are maintained to authorized widths of 500 ft by permeable groins or pile dikes supplemented by dredging. Channels are normally dredged to the authorized depth of 35 ft plus 2 ft of overdepth, but throughout much of the year controlling depths of 30 to 33 ft occur on many of the bars. In recent years, funds have been available for advanced maintenance, and some of the bars have been dredged to 40 or 42 ft and have maintained authorized project depths over a longer period.

Existing Corps of Engineers project

7. The original project for improvement of the Columbia and lower Willamette Rivers from Portland to the sea was approved by the Chief of Engineers 21 May 1877. Initial funds for the project were made available by the River and Harbor Act of 18 June 1878. The project contemplated obtaining a depth of 20 ft at low water by means of dams, dikes, and revetments.

a. This project was modified by the River and Harbor Act of 13 July 1892 (H. Doc. 38, 52d Cong., 1st sess.) to provide for a channel 25 ft deep at low water.

b. The River and Harbor Act of 13 June 1902 adopted a further project for securing, by contraction works and dredging, a 25-ft channel to the sea (H. Doc. 673, 56th Cong., 1st sess.).

c. The River and Harbor Act of 25 July 1912 (H. Doc. 1278, 61st Cong., 3d sess.) authorized a 30- by 300-ft channel from Portland to the sea.

d. The River and Harbor Act of 3 July 1930 (H. Doc. 195, 70th Cong., 1st sess.) authorized the existing 35- by 500-ft project from Portland to the sea.

Channel, 35 by 500 ft

8. By 1930 a total of 92 dikes had been constructed with a total length of 112,420 ft. In addition, dike extensions prior to 1930 amounted to 4560 lin ft. Since 1930, when the 35- by 500-ft channel was authorized,

45 additional pile dikes with a total length of 21,543 ft have been built. In addition, dike extensions built after 1930 amounted to 6755 lin ft. A program of 5-ft advanced overdepth dredging was initiated in 1958 in order to provide full project dimensions on a year-round basis. The additional 3 ft of dredge cut allows the shoaling pattern to continue for a full year and retain full project dimensions.

9. Bars and water depths from Vancouver to the sea for the period July 1960 to June 1961 are shown in table 1.

Table 1

Shoals and Water Depths, Vancouver to the Sea, 1960-61

Shoal	Water Depths in 1960, ft						Water Depths in 1961, ft				
	July	Aug	Sept	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May
Desdemona	35	35	35	35	35	35	35	35	35	35	35
Tansy Point	35	34	34	35	34	34	32-1/2	32-1/2	32-1/2	32-1/2	32-1/2
Astoria Range	32-1/2	31	31	35	35	35	33	33	31	31	31
Tongue Point Range	33-1/2	33-1/2	32	35	33-1/2	33-1/2	31-1/2	31-1/2	31-1/2	31-1/2	31-1/2
Harrington Point Range	35	35	33	33	33	33	33	31	31	31	31
Miller Sands	34	34	31	31	32	32	30	30	30	30	30
Pillar Rock Lower	32-1/2	32-1/2	32-1/2	31	33	33	31	31	31	31	31
Pillar Rock Upper	34	34	34	29	34	34	31	31	31	31	31
Skamokawa	31	31	31	32	32	32	31-1/2	31-1/2	31-1/2	31-1/2	31-1/2
Puget Island	34	34	32	32	35	33	31-1/2	31-1/2	31-1/2	31-1/2	31-1/2
Wauna	32	30	30	35	35	35	35	35	35	35	35
Driscoll Range	31	31	32	35	35	35	34	34	34	34	34
Westport	33	31-1/2	35	35	35	35	35	35	35	35	35
Eureka	32	32	32	32	35	35	35	35	35	35	35
Gull Island	33	33	33	33	35	33-1/2	33-1/2	33-1/2	33-1/2	33-1/2	33-1/2
Stella	33	33	29	35	35	35	35	35	35	35	35
Fisher Island	32-1/2	32-1/2	30	35	35	35	34	34	34	34	34
Walker Island	35	35	32	35	35	35	34	34	34	34	34
Barlow Point	35	35	34-1/2	35	35	35	30	30	30	30	30
Slaughters	32	32	30	35	35	35	32	32	32	32	32
Cottonwood Island	33	33	35	35	35	33	33	33	33	33	33
Kalama Lower	35	35	35	35	35	35	35	35	35	35	35
Kalama Upper	33	33	30	30	30	32-1/2	30	32	32	32	32
Martin Island	33	33	29	32	35	35	33	33	33	32	32
Martin Island Range	31	31	29	28	35	35	31	31	31	31	31
St. Helens	35	35	35	35	35	33-1/2	29	33	33	33	33
Warrior Rock	35	35	34-1/2	35-1/2	35	35	33-1/2	33-1/2	33-1/2	33-1/2	33-1/2
Henrici	33	33	32	32	32	32	33	33	33	33	33
Willow Lower	35	35	35	34-1/2	35	35	35	35	35	35	35
Willow Upper	33	33	31	31	35	35	32-1/2	32-1/2	32-1/2	32-1/2	32-1/2
Morgan Lower	35	35	33-1/2	33-1/2	33-1/2	33-1/2	34-1/2	34-1/2	34-1/2	34-1/2	34-1/2
Morgan Upper	34	34	32	32	32	32	33	33	33	33	33

Channel, 46 by 600 ft

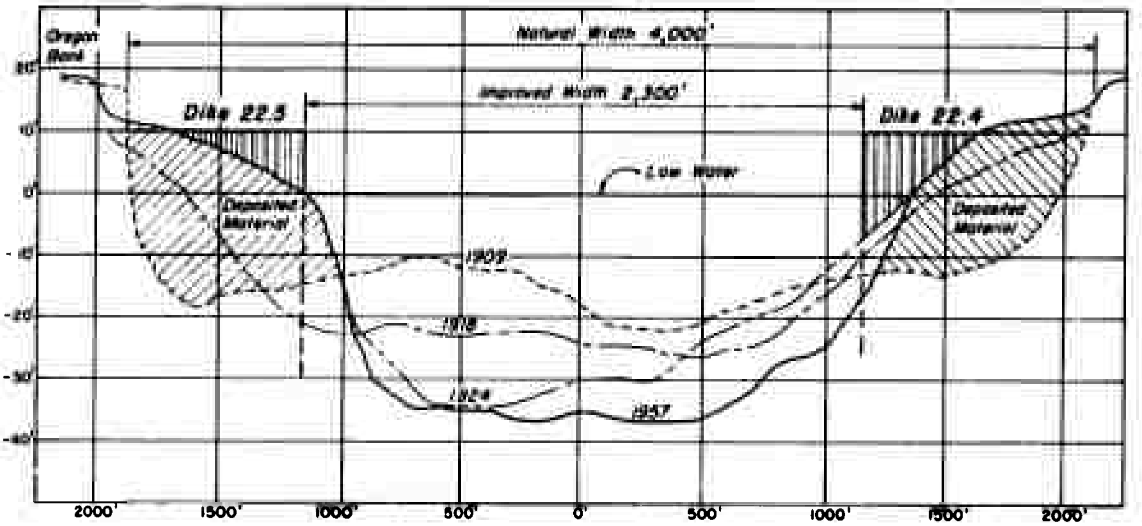
10. The 40- by 600-ft channel recently recommended to Congress will require the excavation of 60,000,000 yd and annual maintenance dredging of 14,000,000 yd. Annual maintenance on the present 35- by 500-ft channel is 9,000,000 yd, and to maintain project depth year-round it is estimated this would increase to 11,000,000 yd.

Proposed pile-dike construction

11. The proposed pile-dike construction considers the building of

29 new dikes and extension of 14 existing dikes as an essential feature in the program of maintaining the increased channel dimensions. Dredging alone cannot insure that project dimensions will be available for a full 12 months. This requires control of a bar by pile-dike construction where project dimensions, initially obtained by dredging, are maintained in large part by natural scouring action.

12. Improvement in bar dimensions as a result of pile-dike construction is shown in fig. 1, and typical pile dike as used on the Columbia River is shown in fig. 2.



COLUMBIA RIVER AT HENRICI BAR - MILE 91

NOTE:
Dike 22.4, 840' long, built 1918, extended 300' in 1932
Dike 22.5, 900' long, built 1918

Fig. 1. Improvement by pile-dike construction

Problems

13. The principal problem is, of course, maintaining the channel in a fixed position to project dimensions. Specific problems are:

a. Reduce maintenance dredging and provide authorized depths, by more effective location, spacing and number of pile dikes.

b. Improve pile-dike design.

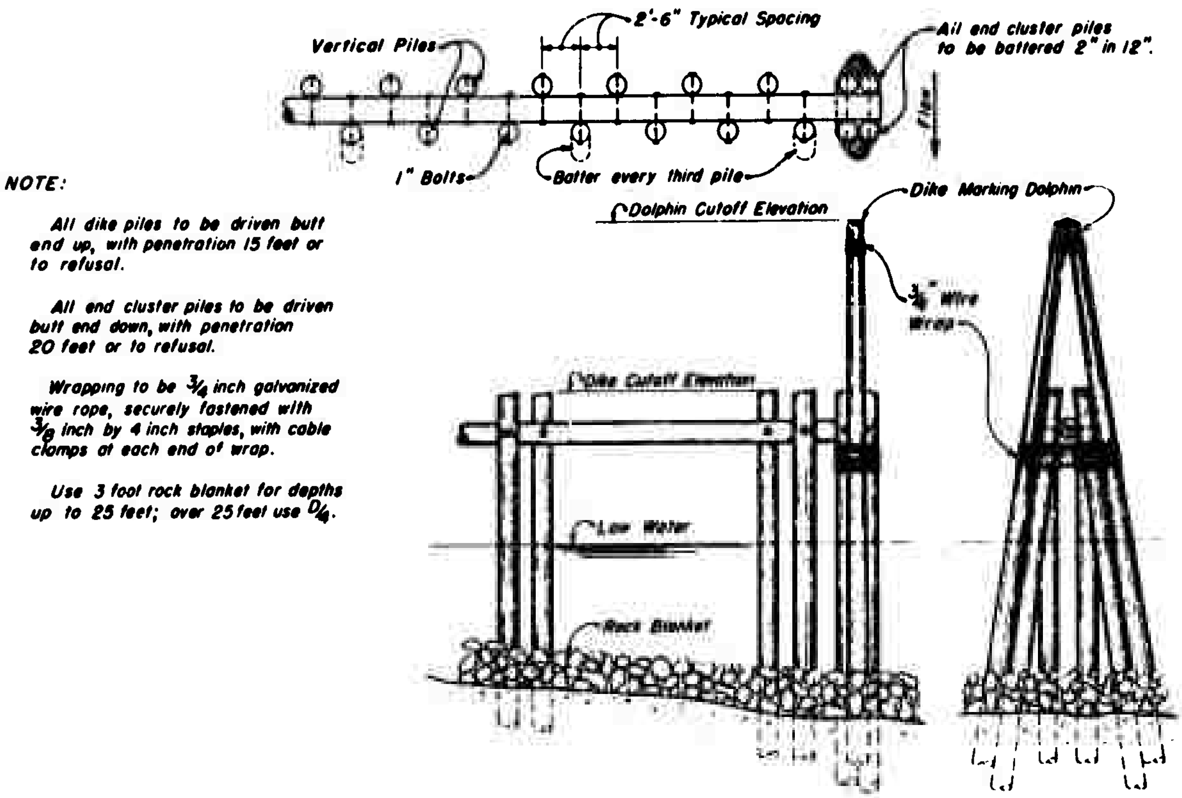


Fig. 2. Proposed pile-dike construction, Lower Columbia River

Table 2

General Data for Columbia

Drainage area	259,000 sq mi
River length	1,200 mi
River fall (total)	2,640 ft
Annual runoff (avg)	178,000,000 acre-ft
Annual flow (avg)	240,000 cfs
Maximum discharge	1,260,000 cfs
Minimum discharge	37,000 cfs
Minimum discharge (reg)	75,000 cfs
<u>Tides at mouth, datum mllw</u>	
Maximum	+10.5
Minimum	-2.5
Tidal reach	140 mi
Low-water gradient	0.05 per mi
Velocity (maximum) at Vancouver	7 ft/sec
Velocity (minimum) at Vancouver	3 ft/sec
Channel materials	(SP) sand

(Continued)

Table 2 (Concluded)

<u>Tides at mouth, datum mllw (Cont'd)</u>	
Number of bars (Puget Island to Vancouver)	16
Number of pile dikes	150+
Project authorization, Vancouver to sea	35 × 500 ft
Annual maintenance dredging	
Estuary	2,000,000 yd
Vancouver to estuary	7,000,000 yd
<u>40- by 600-ft channel</u>	
Initial dredging required	60,000,000 yd
Annual maintenance dredging	14,000,000 yd
New pile dikes or extensions	43

CHAPTER VII
RESEARCH ON CHANNEL STABILIZATION

Recommended Research on Channel Stabilization Problems

by

J. B. Tiffany*

Recommended research

1. Table 1 outlines "Suggested Headings and Subheadings for Review of Literature on Rivers and River Regulation," which was prepared originally with specific reference to the Mississippi River. It is suggested that the items included in this outline might be used not only in connection with reviews of literature on this subject, but as a checklist for determination of the areas in which sufficient information is lacking and therefore in which research might be needed.

2. Table 2 outlines "Suggested Future Specific Studies and Investigations," which also was originally prepared with reference to the Mississippi River. It is suggested that this outline may also be used as a checklist for needed research, not only for the Mississippi River but for the other large rivers presumed to be under the purview of the Committee on Channel Stabilization.

3. Specific areas in which the writer believes that research is needed are discussed in the following paragraphs.

4. Principles of river alignments. Among the most important subjects with which an engineer responsible for river improvement should be familiar are the basic principles of the alignment of the river in question. Some information is available on this subject, but it is believed that most of the knowledge is empirical, based on observation and experience only, and lacking a theoretical basis. We know qualitatively that the larger the river (from the standpoints of discharge and other basic factors), the larger the bends which the river tends to form. However, I believe that we do not have quantitative knowledge as to the relation between the basic dimensions of a river and its natural meander pattern.

* U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

5. Furthermore, we do not know what relation exists between one given bend in a river and the resulting shape of bends downstream. It is realized that the types of material composing the banks of a river have much to do with the general meander pattern which develops and with the shape of individual bends. However, it also appears to the writer that the hydraulic conditions produced in one bend, or in one or two successive bends, must be a major factor in determining the shape of the next one or more bends downstream. Specifically, the writer is inclined to believe that a given shape of bend produces energy (and turbulence) distributions which have marked effects on the shapes of bends downstream. To put it simply, it appears that the shape of one bend "begets" the shape of bends downstream.

6. It is recommended, therefore, that a research study be undertaken to determine such basic principles.

7. Hydraulic forces. A considerable portion of the writer's paper, Review of Research on Channel Stabilization of the Mississippi River, 1931-1962, is devoted to the subject of the importance of the hydraulic forces in a river and methods of computing or measuring such forces. The writer believes that this is a subject of major importance to all of the proposed improvement works in our major rivers. The classic routine in the design of all civil engineering structures, except those placed in rivers, includes first the determination of the forces which the structure in question will be required to withstand. We know qualitatively that hydraulic forces in the river are tremendous, particularly during high stages, and we know that such forces have been large enough to destroy almost every type of structure which has been built in our rivers. However, except for the fragmentary information contained in the writer's paper referred to above, it is believed not only that very little has been done on this important subject, but that the importance of it has been almost entirely overlooked in our so-called design of river improvement or bank stabilization structures.

8. It is, therefore, strongly recommended that a program of research be undertaken to determine the magnitude of the hydraulic forces against which we have to design.

9. Sediment transport. The writer believes that this subject is

also of paramount importance to practically all if not all of our research improvement programs. Considerable information is available on this subject, but it is believed that the information available should be reviewed and documented, and that additional information probably is required. In most of the rivers under the purview of this Committee, that part of the sediment load usually considered as the bed load--that is, the sand or larger components--is the one of most importance, but the one of which the least is known. It is believed that information as to the finer sediments, which usually remain in suspension until the river nears its mouth, is of less significance to the rivers with which we are concerned.

10. As brought out in the writer's paper referred to above, a major difference--one or two "orders of magnitude"--exists between some concepts held as to the amount of bed load in a river as compared with the concepts of others which the writer believes are based on actual measured data.

11. The importance of this subject to bank stabilization, and to the efficiency and permanence of dredging, is believed clear. The writer recognizes that the actual measurement of the rate of bed-load movement is extremely difficult if not downright impossible. However, he believes that there is a wealth of information available in the records of the District Offices from which order-of-magnitude conclusions can be derived as to rates of bed-load movements. Such information would include the hundreds of hydrographic surveys which have been made, and which could be analyzed by scour-and-fill computations, to arrive at much-needed information.

12. It is therefore recommended that a program of research in this subject be initiated, possibly to include first the analysis of existing data of the type described above.

13. <u>Composition of riverbeds and riverbanks.</u> This subject is believed to be of almost obvious importance. Certainly there is a great difference in the characteristics of rivers flowing between sand banks and of rivers flowing between clay or rock banks, with many complicated situations in between those extremes. The writer does not visualize any general research on this subject, but rather points out the necessity for knowledge on this subject for each river and for each problem reach of each river. This matter has been found to be of overriding importance in the consideration of improvements of the Mississippi River.

14. <u>Stability of banks against caving.</u> This subject is, of course, closely related to the preceding subject. On the Mississippi River, for instance, it has been found that bank caving may occur from a large number of causes, ranging from minor shear failures to larger liquefaction (flow slide) failures which have involved the rapid movement of as much as 4,000,000 cu yd of bank and overbank in a matter of hours or less. This is a matter of great concern on the Mississippi River, but the writer does not know whether it may be of similar concern in the other large rivers.

15. The writer's recommendation with regard to this subject is that programs of research, investigation, and analysis be initiated on each river concerned to determine the manner of bank failures and means for their prevention or alleviation.

16. <u>Evaluation of effectiveness of presently used revetments.</u> One of the principal types of structures used in channel stabilization, of course, is bank revetment, and various types of revetments have been developed in the various rivers of concern to this Committee. An analysis should be made of the effectiveness of all these types of revetments under the conditions to which they are subjected. Obviously, the objective of such an analysis should be to develop better revetments to replace any present types which may be found to be less than completely satisfactory and/or economical. Such an investigation should include at least the following items.

<u>a</u>. <u>Instruments to determine the existence and condition of revetments.</u> Once a revetment has been laid it is seldom possible to determine its condition after it has been subjected to attack, nor even to determine whether it still remains in place. An instrument has long been sought which could distinguish between a revetment and the natural bottom of the river, but until recently nothing satisfactory has been developed. Reference is made to the paper cited in paragraph 7, paragraph 119 of which describes tests recently made by the New Orleans District on a device called the "sonar pinger." This instrument seems to offer great promise for meeting this long-felt need, and it is recommended that steps be taken to continue this development. Another step which is recommended would be the making of periodic underwater surveys, using divers.

<u>b</u>. <u>Studies of actual revetment failures.</u> It is well known that a considerable number of failures occur in our revetments each time they are subjected to attack, but it is believed to be seldom that the exact reasons for the failures are

determined. It is recommended that programs be established in all the District Offices concerned to analyze revetment failures, in detail, and to document these analyses. These analyses should include the answer to such questions as: Why did the failure occur? What hydraulic forces and turbulence were involved? What was the possible effect of the alignment of the river leading into the revetment? What types of materials were in the banks of the failed areas? What were the bank slopes? Did appreciable "river attack" exist elsewhere along the revetment than where the failure occurred? If so, was there any difference between the intensity of the attack? What type of failure was it--shear, flow slide, etc.?

c. Studies of periodic scour and fill. Most alluvial rivers have a cycle of scour in the bends during rising and high stages, and fill during subsequent low stages. A study should be undertaken to relate this phenomenon to the design of revetments.

d. Strength of revetments. A study should be undertaken to determine the necessary strength of component parts of revetments, as well as the basic weight (or thickness) of the concrete or riprap required to resist the hydraulic forces in the river. This study should include an analysis of the methods of anchoring revetments.

e. Effect of revetment placement procedures on stability. It is believed that many revetments (particularly on the Mississippi River) have failed or partially failed because of inadequate methods of placing the revetments. It is recommended that this subject be among those investigated.

17. Evaluation of effectiveness of dikes, groins, etc. A great variety of dikes, groins, etc., have been used in different rivers, or even in different parts of the same river. It is recommended that an analysis be undertaken to determine which have been effective and which have been ineffective, and the reasons therefor.

18. Effectiveness of dredging and soil disposal. It is believed that a very careful analysis should be made of the dredging procedures used in the rivers of concern, since it is considered possible that at least some of the dredging is misdirected and/or ineffective. This analysis should include studies of methods and locations of disposing of the dredge spoil.

Concluding comments

19. It will be noted that all of the above-suggested areas of research involve full-scale (prototype) observations and/or analyses of

information from the rivers themselves. No recommendation has been made herein as to any research work on models or in a laboratory.

20. However, in the next paper, "Suggested Studies and Investigations by Committee Members," recommendations by Mr. G. B. Fenwick of the Waterways Experiment Station cover research work which could be conducted in a laboratory. It is suggested that a coordinated program of research be developed, which would make a balanced use of field and laboratory approaches.

Table 1

Suggested Headings and Subheadings for Review of Literature on Rivers and River Regulation

A. Natural river phenomena and characteristics

1. Geology
   a. History of river and its development through geologic time
   b. Characteristics of the present river
   c. Migration of meanders (including cutoffs, etc.)
   d. General nature of bed and banks
   e. Bed load and sediment load
   f. Groundwater problems
   g. Scour and deposition patterns
2. Soils
   a. Characteristics of the bed and banks of the river (see 1d above)
   b. Soil properties, and their reference to modes of failure
   c. Stability of banks, and modes of failure (shear, flow, etc.)
   d. Groundwater and seepage factors (see 1f above)
   e. Field exploration methods and their applicability to bank failures
   f. Laboratory test methods and their applicability to bank failures
   g. Empirical methods of predicting bank stability*
   h. Rational methods of predicting bank stability*
3. Hydraulics*
   a. Meandering (theories, and practical knowledge as related to other factors--soils, bank erosion, slope, discharge, etc.) (see 1c above)
   b. Hydrology (stages, discharges, flood frequencies, etc.)
   c. River alignment (theory and practical knowledge, as related to other river characteristics)
   d. Cross sections

(Continued)

* Perhaps more important than any of these items above is the combination of these factors--the hydraulics forces as the causative factor, and the soil strengths as the resisting factor.

Table 1 (Continued)

e. Velocity patterns

f. Turbulence (energy relations, boundary layers, tractive force, uplift, friction; velocity fluctuations, pressure fluctuations; transmittal of pressure fluctuations through saturated sand)

g. Effect of curvature of channel on depth

h. Sand waves (effect on stage-discharge relation)

i. Water temperature (effect on stage-discharge relation)

j. Salinity (effect on currents and sediment-movement characteristics)

4. Sediment movement

a. Character of bed and banks (see 2a above)

b. Quantity of suspended sediment in transport

c. Quantity of bed load in transport (see 1e above)

d. Bed-load formulas (theoretical and empirical)

e. Manner of bed-load movement (saltation, etc.)

f. Scour and deposition patterns (see 1g above)

B. Instrumentation for

1. Velocities, velocity fluctuations, velocity directions
2. Pressures, pressure fluctuations
3. Discharges
4. Hydrographic surveys
5. Bed-load measurements
6. Sediment-load measurements
7. Detection of underwater revetments and structures
8. Turbidity
9. Density
10. Salinity
11. Aerial photography
12. Aerial sounding

C. Control measures (theory and practical use--excluding construction methods) (see D below)

1. Levees
2. Cutoffs

(Continued)

Table 1 (Concluded)

3. Revetments
4. Dikes
5. Groins
6. Dredging and dredge spoil disposal
7. Diversions
8. Floodways

D. Construction and maintenance of control measures

1. Bank protection
    a. Stabilization of banks
    b. Bank grading (above and below water surface)
    c. Upper bank protection (riprap, asphalt, other)
    d. Underwater revetment (concrete mattress, willow mats, riprap, concrete or asphalt blocks, mass asphalt, other; possible new designs)
    e. Subsurface (soil) exploration
    f. Prediction of potential failure areas, and remedies therefor
2. Other improvement methods (dredging, cutoffs, etc.) (To be divided into proper subheadings)

E. Laboratory aids

1. Hydraulic models
    a. Fixed bed
    b. Movable bed
2. Electrical analog models
3. Soils laboratory testing and analysis
4. Materials testing

Table 2

Suggested Future Specific Studies and Investigations

I. Hydraulic forces acting on revetments (basic design factors)

   A. Velocity forces, and velocity fluctuations

   B. Variations in pressure, possibly resulting in uplift

   C. Linear extent (horizontal and vertical) of fluctuating forces and possible uplift of revetment. How does buoyant weight of revetment compare with uplift forces?

   D. Transmission of pressure changes through pore water in sand under revetment

II. Evaluation of effectiveness of articulated concrete revetments used at present

   A. Field determination of existence and location of revetments

      1. Sonar "Thumper"

      2. "Pinger"

      3. Seismograph

      4. Other methods

   B. Studies of actual revetment failures (in sufficient detail to determine reasons)

   C. Detailed studies of frequent failures at upstream ends of new revetments

      1. Why do they occur? Hydraulic forces involved? Turbulence? Alignment? Bank slope? Type of materials in bed and bank?

      2. Does appreciable "river attack" exist elsewhere along a newly revetted bank than where failures occur? If so, is there any difference in intensity of attack? If not, was revetment required originally in nonfailure parts of bank? Is timing of revetment placement worth considering?

      3. After upper-end failure has been repaired, and the revetment eventually held in place, does it remain in place because it has been made "stronger" (more resistant to "attack")? Or has river "attack" moved downstream? If so, was the revetment patch necessary?

   D. Regarding the scour (30 to 80 ft vertically) which usually occurs (in the Mississippi River) in the first high water after a revetment is placed:

(Continued)

Table 2 (Continued)

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1. Does scour occur--or is it aggravated--because the revetment is there? Or would the scour occur even though the revetment were not there? Does the size of openings have any effect on whether scour would occur? Does scour result principally from leaching through the openings, or does it progress inward from the outer end of revetments?

2. What happens to the revetment when the river bottom scours 30 to 80 ft? Where is the revetment then?

3. During the subsequent 30- to 80-ft shoaling during low water, what happens to the revetment, assuming it is still in place? Is it buried under sand? If so, is it effective during the next high water as bank protection? If "yes," how? If "no," is that part of the revetment which is not buried the only effective bank protection? If so, why do we extend revetments to the river thalweg if only the part immediately adjacent to and on the bank is in contact with the water?

4. Summing up 3 above, if the outer ends of all revetments scour with the first high water, why is that part of any value, and therefore why do we build them to the thalweg? Or in other words, if extending the revetments to the thalweg in order to prevent scour from the thalweg to the bank is unsuccessful (and it appears to be), why do we build revetments any longer than the part that is doing some good?

E. Conclusions we need to reach (from studies of A to D above)

1. Are present revetments as effective as they should be?

2. Are we building revetments to the optimum, economical length?

F. If the answer to II-E-1 or II-E-2 above is "no," what steps are in order?

III. Effect of revetment placement procedures on revetment stability

A. Effect of bank preparation methods

1. Upper bank

2. Subaqueous

B. Effect of irregularities in bank line

1. Horizontal alignment

2. Vertical alignment ("waves," loose material, etc.)

3. Obstructions and debris (trees)

C. Irregularities in revetment itself ("steps," offsets, etc.)

(Continued)

Table 2 (Concluded)

IV. Optimum river alignment

A. Optimum curvature

B. Length of crossings

C. Proper meander pattern

V. Volume of bed load

VI. Flow slides

A. Further study is required of case examples, causes, and methods of predicting <u>susceptibility</u> of banks to flow slides

B. Methods of stabilizing banks to prevent flow slides

C. Of equal importance is determination of <u>hydraulic factors</u> which <u>cause</u> a slide to occur in a bank which is susceptible

VII. Other types of bank failures

A. Need study

B. Geological studies needed in connection with all bank failure studies

Suggested Studies and Investigations by Committee Members

by

R. W. Sauer*

Recommended by Mr. Tiffany:

I. Hydraulic forces acting on revetments (basic design factors)

A. Velocity forces, and velocity fluctuations

B. Variations in pressure, possibly resulting in uplift

C. Linear extent (horizontal and vertical) of fluctuating forces and possible uplift of revetment. How does buoyant weight of revetment compare with uplift forces?

D. Transmission of pressure changes through pore water in sand under revetment

II. Evaluation of effectiveness of articulated concrete revetments used at present

A. Field determination of existence and location of revetments

1. Sonar "Thumper"

2. "Pinger"

3. Seismograph

4. Other methods

B. Studies of actual revetment failures (in sufficient detail to determine reasons)

C. Detailed studies of frequent failures at upstream ends of new revetments

1. Why do they occur? Hydraulic forces involved? Turbulence? Alignment? Bank slope? Type of materials in bed and bank?

2. Does appreciable "river attack" exist elsewhere along a newly revetted bank than where failures occur? If so, is there any difference in intensity of attack? If not, was revetment required originally in nonfailure parts of bank? Is timing of revetment placement worth considering?

3. After upper-end failure has been repaired, and the revetment eventually held in place, does it remain in place because it has been made "stronger" (more resistant to "attack")? Or has river "attack" moved downstream? If so, was the revetment patch necessary?

D. Regarding the scour (30 to 80 ft vertically) which usually occurs (in the Mississippi River) in the first high water after a revetment is placed:

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* Formerly with Lower Mississippi Valley Division, Vicksburg, Mississippi.

1. Does scour occur--or is it aggravated--because the revetment is there? Or would the scour occur even though the revetment were not there? Does the size of openings have any effect on whether scour would occur? Does scour result principally from leaching through the openings, or does it progress inward from the outer end of revetments?

2. What happens to the revetment when the river bottom scours 30 to 80 ft? Where is the revetment then?

3. During the subsequent 30- to 80-ft shoaling during low water, what happens to the revetment, assuming it is still in place? Is it buried under sand? If so, is it effective during the next high water as bank protection? If "yes," how? If "no," is that part of the revetment which is not buried the only effective bank protection? If so, why do we extend revetments to the river thalweg if only the part immediately adjacent to and on the bank is in contact with the water?

4. Summing up 3 above, if the outer ends of all revetments scour with the first high water, why is that part of any value, and therefore why do we build them to the thalweg? Or in other words, if extending the revetments to the thalweg in order to prevent scour from the thalweg to the bank is unsuccessful (and it appears to be), why do we build revetments any longer than the part that is doing some good?

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   A. Effect of bank preparation methods

      1. Upper bank

      2. Subaqueous

   B. Effect of irregularities in bank line

      1. Horizontal alignment

      2. Vertical alignment ("waves," loose material, etc.)

      3. Obstructions and debris (trees)

   C. Irregularities in revetment itself ("steps," offsets, etc.)

IV. Optimum river alignment

   A. Optimum curvature

   B. Length of crossings

   C. Proper meander pattern

V. Volume of bed load

VI. Flow slides

A. Further study is required of case examples, causes, and methods of predicting susceptibility of banks to flow slides

B. Methods of stabilizing banks to prevent flow slides

C. Of equal importance is determination of hydraulic factors which cause a slide to occur in a bank which is susceptible

VII. Other types of bank failures

A. Need study

B. Geological studies needed in connection with all bank failure studies

Recommended by Mr. Fenwick:

a. The first recommendation is to gather field information from several selected rivers that will show three things: (1) river conditions before construction of regulative works; (2) as-constructed details of the structures; and (3) developments in the rivers since construction of the works. Mr. Fenwick thinks such information can be obtained from the various districts concerned. This would be a continuing activity for some years, and would seem to constitute the best possible source of information needed by river engineers. As an example, a new system of dikes has been constructed in the Savannah River to provide a 9-ft channel, and Mr. Fenwick has already obtained plans of the structures as well as a promise from the district of survey data showing their performance in the river so far. He also hopes to visit this site to see and photograph these structures and find out how they are working out. These structures look good, and Mr. Fenwick is interested in their applicability to other similar rivers. In his opinion, documented reports on performances of river structures must replace word-of-mouth advice on how to regulate various rivers. A great deal of information must be available, but remains to be collected, analyzed, and reported.

b. As a second item, Mr. Fenwick sees the need of a laboratory research project concerned with the characteristics of dikes used in river regulation. He has read considerable literature on the subject and has asked opinions of a good many river engineers, and opinions are widely divergent. Among the many questions in need of answers are the following: (1) how does degree of dike permeability affect fill between dikes and scour along the ends; (2) what are the optimum crest elevations of pile dikes and stone dikes with respect to the planes of low, mean, and maximum flow; (3) what are the advantages and disadvantages of horizontal, sloping, and stepped dike crests; (4) what are the advantages and disadvantages of dikes placed normal to flow, slanted upstream, and slanted downstream; and (5) what considerations should govern the spacings between dikes of different types. It is proposed that these and other related matters be studied, not in a model, but in a small laboratory stream with fixed banks and a sand bed. It is realized that many engineers are prepared to give answers already, but there is wide disagreement and little if any documentation of

answers. A laboratory study is needed to isolate the effects of the points in question.

Recommended by Mr. Lindner:

a. Research to ascertain the proper spacing of dikes under various conditions that could have an effect upon the spacing. For example, other conditions being the same, it should be evident that different spacings are required for optimum effectiveness with different current velocities, for with higher velocities the angle of spread of a contracted jet is less than for lower velocities.

b. Closely allied with the foregoing problem is the determination of the angle of spread of the contracted stream. To know this would be helpful in many analytical calculations where contraction is being considered. In studying this phenomenon, velocity traverses will be required, for where contraction has been sudden, the high velocity jets tend to be very localized in the vicinity of the contracting structures, with the velocities in the remainder of the cross section very little affected. Thus where the purpose of contraction is to maintain an increased channel depth, the purpose may not be attained if the increased velocities are so localized that only holes are dug near the ends of the contracting works. This leads to the next item of research.

c. Ascertain whether the localizing of high-velocity jets at the end of contracting works can be prevented by gradual contraction, and how it should be done under various conditions to accomplish this end. The main purpose here is to provide better channel maintenance by increasing the velocity generally across the channel rather than principally at the end of a dike with the result that a hole is eroded there which threatens the integrity of the dike and does not provide a suitable navigation channel. What is the optimum rate of contraction from the standpoint of effectiveness and economy? Obviously, of course, there is no economy if the contraction works are ineffective.

d. Study the development of scour holes at the ends of dikes. Obtain the shapes under varying velocities, angles of dikes with the current, etc. Ascertain the limiting depths of such scour holes. It would appear that blanket protection would be most effective if placed in the shape of and at the location of the anticipated scour hole. How far beyond the limits of the anticipated scour hole should the blanket extend?

e. It is the general practice to place a protective blanket (usually quarry-run stone or the like) along a permeable dike. What would be the scour pattern along the dike if this protection were omitted except, of course, at the head and foot of the dike? Would the scour be such as might cause failure of the dike? Will the sand deposit upstream from such a dike? It would seem that it would, so that eventually the flow through the dike would be diminished with the result that the erosion cavities would at least partially fill. It seems that this could be determined from small-scale tests. If it is then indicated that under certain conditions the lateral rock may be omitted, much saving will result.

f. Ascertain the spacing of piles in a permeable dike that will be effective (and possibly most effective) with different sizes of bed-load material.

g. Perhaps some tests could be made to determine the best way to tie the dike into the bank to prevent flanking.

h. It seems to me we need to know more with regard to average channel velocity required to move sand of varying grain sizes. We also need to know the maximum average velocity that will not move various sands. I know that so-called tractive force is usually spoken of in this regard, but that has been theoretically derived and we need proof of its applicability. Velocity represents one of the elements in the tractive force equation, anyway - slope. It will be much simpler if sand movement can be related only to mean channel velocity in channels with normal vertical velocity curves. Since velocity represents energy and it takes energy to move anything, perhaps sand movement can be related to velocity. It has been common engineering practice to so assume.

i. In considering contraction works, engineers often speak of overcontractions and as a result, doubtless, often contract less than desirable to avoid overcontraction. Overcontraction, of course, could cause excessive velocities in the navigation channel, erosion of the opposite bank, or failure of the contraction works due to scour hole erosion at the end of the dikes. If the gradual contraction mentioned above is effective, the latter could be avoided. But since this phenomenon is apparently an unknown among designers, it should be investigated.

j. Attempts should be made to ascertain means of terminating blankets and revetments so that erosion will not occur at the upstream and downstream ends. Of course it would be desirable to prevent erosion at the channel limits of these structures also, but that might be a more difficult problem. It would seem that the problem is to prevent increase in turbulence at these points. Perhaps some smoother type of structure would minimize erosion. Anyway it appears that the problem merits research.

k. Determination of current directions above, below, between, and along dikes under various conditions, and water elevations and slopes also.

Recommended by Mr. Bondurant:

a. In regard to channel stabilization research projects, Mr. Bondurant believes that one of the most important should be an investigation to determine the characteristics of meander dimensions with respect to discharge and sediment load.

b. Mr. Bondurant also suggests an investigation of the effects of permeable control structures. With respect to pile dike we have learned by experience; however, with respect to the jacks as used on the Rio Grande there is very little design experience available except that the Kellner people do seem to have excellent judgment in their recommendations. The Bureau of Reclamation has made one laboratory test which he considers only a beginning.

c. Mr. Bondurant is particularly interested in the turbulence effects which tend to reduce the velocity adjacent to the structure and thus encourage the flow to turn versus the head loss causing a reduction in velocity of flow passing the structure.

Recommended by Mr. Bubenik:

a. For purposes in the Northwest, pile-dike design, including spacing and orientation, needs further study.

b. Also, a prototype study of currents and velocities immediately adjacent to the river bottom is considered essential to a better understanding of bed-load movements.

Recommended by Mr. Isaacs:

None until research and evaluation of literature have been completed.

Recommended by Mr. Douma:

a. It seems that research on channel stabilization can be divided into the following classifications:

(1) Literature search and documentation.

(2) Collection and evaluation of available data.

(3) Prototype research.

(4) Laboratory research.

b. Mr. Douma thinks the Committee should establish a comprehensive long-range research program, possibly divided into the above-suggested categories, but the immediate goals for the next few years should be geared to its capability and rather limited funds. It may be wise to limit the Committee's activities in research to the first two categories until specific problems are more clearly defined, after which it should be possible to more effectively establish prototype and laboratory research.

Recommended by Mr. Sauer:

a. Award a contract for research and development of a chemical process to stabilize banks by an injection method which will produce a "hardpan." The development of such a process would permit predetermination of an optimum channel alignment, establishment of safe bank slopes, and protection against failure from channel deepening and undermining which frequently occurs following placement of normal revetments. Such a development would be of great value even if its application was limited to porous materials.

b. Develop criteria for utilization of pile and stone dikes as related to movement of material, slope, and general river characteristics.

c. Using data to be collected from Districts, make a study to develop dimensionless parameters, e.g. width/depth ratio, sinuosity, rate of bank caving, etc. From numerous partial sets of data, determine statistical correlations (simple and multiple regression analyses). The end product would aid in understanding, and in defining the mathematical basis for, relations between parameters.

Suggested approach for establishing research projects by Mr. Sauer

The suggested approach is outlined as follows:

I. Obtain data on alluvial rivers which have been economically and successfully stabilized (see table 1).

II. Initiate research and development of new materials or new uses of existing materials which may be effective for bank stabilization without awaiting answers to the basic causes of bank caving.

III. Develop equipment which will positively locate revetment after placement and which will permit checking its condition at subsequent times.

IV. Set up method of reporting and cataloging failures.

V. Institute series of dike laboratory tests to determine effect of:

a. Permeability.

b. Crest elevation with respect to low and medium stages.

c. Vertical profile.

d. Angularity with respect to flow.

e. Spacing of dikes in dike field.

VI. Basic research.

a. Hydraulics.

b. Meander dimensions with respect to discharge and sediment load.

c. Effect of nonuniformity of underwater grading.

d. Effect of nonuniformity of curvature alignment, including offsets in alignment.

e. Degree of "tightness" of revetment required to:

(1) Prevent erosion of foundation sands.

(2) Prevent erosion of silts and clays.

It is felt that this outline of approach will accommodate most if not all the individual research projects proposed. The order of priority as set forth was dictated by my feeling that improvements to our present methods of stabilizing banks must be attempted while we are researching basic data.

It must be remembered that many millions of dollars are spent annually on construction and maintenance of stabilization works. Any early advancement of the "art" should be considered worthy of the effort.

Fund availability will obviously dictate the extent of yearly programs.

Table 1

Alluvial River Data

A. Optimum river alignment. Obtain and analyze for pattern, data from good reaches of rivers which require minimum maintenance

a. Rivers with maximum discharge up to 100,000 cfs

b. Rivers with maximum discharge between 100,000 and 500,000 cfs

c. Rivers with maximum discharge between 500,000 and 1,000,000 cfs

d. Rivers with maximum discharge in excess of 1,000,000 cfs

B. Evaluate successful methods of channel stabilization. This evaluation should be made in terms of river sizes listed in A above. Furnish design, and before and after surveys when available. List design types that failed to perform and give apparent reasons when known. Furnish data on:

a. Bank stabilization method (subaqueous and upper bank)

b. Permeable dikes

c. Rock dikes

d. Other

C. Draw tentative conclusions from A and B, and using these as design criteria, stabilize a test reach of river for verification purposes

Hydraulic Models for Study of River Sedimentation Problems

by

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Abstract

This paper reviews the state of knowledge concerning the hydraulics of natural streams and related sedimentation problems. It points out the difficulty encountered by the river engineer in the solution of sedimentation problems by analytical means and the reasons why the development of basic laws and principles from experience in the field has not been practical.

The opinion is expressed that the advancement of the principles involved in river engineering and the solution of many sedimentation problems will have to depend for the most part on laboratory studies correlated with results of field investigations. The types of studies required are mentioned and the types of models used for these studies are described. One section of the paper is devoted to the design, verification, and operation of the movable-bed model.

Movable-bed model studies in connection with the canalization of the Arkansas River and some of the results which could be applicable to alluvial rivers in general are discussed. The paper also describes an investigation in progress to determine the effects of water temperature on the movement of bed material and bed formations and includes preliminary results indicated by the study so far.

1. Developments within natural streams involving sedimentation are affected by many factors, some of which are interrelated and extremely complex. The struggle toward the regulation, stabilization, and improvement of natural streams for water supply, soil conservation, navigation, and flood control has been going on for centuries in all parts of the world. In spite of the vast effort and experience, few basic principles have been developed which can be used by the engineer in the design of river structures, and little is known as to why some structures are effective and some are not. The evaluation of conditions in the field is extremely difficult because of the variation in geometry from one reach to the next, the variation in flow conditions, and the impracticability of comparing the effectiveness of various types and layouts of structures under the same conditions. Efforts have been made to obtain data on the performance of certain structures in the field, but very little of the available data has been

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evaluated and documented. In most cases, the data obtained have not been in sufficient detail to permit a reasonable evaluation of conditions before and after the installation of the improvement works.

Investigations of sediment problems

2. The study of the mechanics of sedimentation has been the object of many research projects. A large amount of information is available covering the works of various experimenters and the evaluations of the results of such studies have led to the development of several sedimentation formulas. Since these formulas were derived mostly from flume studies involving two-dimensional flow, they can be applied only in a general way to the solution of problems in natural streams. There is additional need for research involving the factors affecting the movement of sediment, but the ultimate solutions of many practical problems will require research in the field and research with models of natural streams reproducing three-dimensional flow. Hydraulic models have led to the development of some of the basic factors affecting the movement of sediment and the meandering of alluvial streams. Model studies have also pointed out some of the characteristics of structures affecting their performance in alluvial streams and have led to a better understanding of some of the processes involved in the development of channel configurations. However, the knowledge of the hydraulics of natural streams and of the sedimentation processes is not sufficient to permit the solution of many problems by analytical means with any degree of confidence. It is believed that the advancement of the science of river engineering, the optimum solution to many sedimentation problems, and the answers to many questions regarding the performance and effectiveness of various types and arrangements of structures will have to be based for the most part on the results of model studies correlated with results of field investigations.

3. Field investigations to be of value in the solution of sedimentation problems should be planned for that purpose and should be one of the major objectives of the project. When construction is involved, the investigation should be started in sufficient time to permit an evaluation of developments in the reach with low and high flows and should be continued during and after construction until the effects of the works involved have been definitely established.

Hydraulic models

4. Hydraulic models used for the study of sedimentation problems are either of the fixed-bed type or of the movable-bed type. The study of problems in the fixed-bed type of model is based on the direction of movement of lightweight, plastic material or dye, and current direction and velocity measurements. This type of study has certain advantages in that such models can be designed to obtain hydraulic similitude, are easier to adjust and less expensive to construct and operate, and results can be obtained within a comparatively short time. However, fixed-bed models are generally suitable only for the study of local problems in relatively stable streams involving the movement of small quantities of sediment.

5. The model generally used for the study of problems in alluvial streams is of the movable-bed type having fixed banks and overbank with the bed molded of a granular material such as sand, crushed coal, granulated plastic, etc. Erodible banks have been used to reproduce sections of banks where active caving is present in the prototype stream or where bank caving can be expected from the installation of the improvement plans to be tested; however, the use of erodible banks is usually not considered practicable because of the variations in the erodibility and other characteristics of the materials forming the banks of the natural stream and the difficulty of reproducing the conditions in the model with any degree of accuracy even if enough prototype data were available. When bank caving is a factor in the development of the stream, its effect can usually be simulated artificially in the model without compromising the ultimate results.

Movable-bed models

6. Design. The design of a movable-bed model must consider the characteristics of the stream involved, the problem or problems to be investigated, and the requirement that the material forming the channel bed be moved in simulation of the sediment movement in the prototype. Other factors which also affect the design of the model are space and facilities available, and cost and time limitations. The scales selected for movable-bed models are generally too small to provide hydraulic forces sufficient to move material of a practical size and specific weight. The forces required to obtain general movement of the bed material are obtained by distortion of the linear scales and/or by use of supplementary slope.

Distortion which involves the use of a vertical scale larger than the horizontal scale provides greater model depths and slope. Supplementary slope provides a tilt to the model in addition to that resulting from the linear scales, thus increasing the total model slope by a fixed amount. Distortion of the linear scales and supplementary slope tend to affect the relation of velocity, width-depth ratio of the channel and curvature, and consequently tend to affect the distribution of energy within the channel. Also, distortion produces changes in the size and shape, which in turn affect the performance of certain types of structures. Because of these effects, distortion and supplementary slope should be maintained as small as feasible. The effect of distortion on the performance of some structures such as sills or stilling basins can usually be eliminated by using the vertical scale to reproduce certain horizontal dimensions, particularly the dimensions in the direction of flow.

7. Verification. Complete similarity between a movable-bed model and its prototype can seldom be obtained because of inherent distortions which are incorporated during model design and the impracticability of reducing the grain size of the bed material to the scale of the model. Because of the dissimilarities mentioned, the degree of similitude attainable in these models cannot be established by mathematical analysis. The reliability of a movable-bed model is usually based on the model verification. The normal verification is an intricate process of adjusting the various hydraulic forces and model operating technique until the model demonstrates its ability to reproduce with acceptable accuracy the changes in bed formations which are known to have occurred in the prototype during a given period. The successful verification of the model is used to establish the time and discharge scales, the rate of introducing bed material for each flow reproduced, model operating technique, and the degree of similitude which can be expected. The verification involves considerable time and effort, careful observation and analysis of developments, and the exercise of engineering judgment in the evaluation of the factors affecting development in the prototype and model. The degree of similitude attainable and the time required for model verification depend upon the amount and type of prototype data available, the scales of the model, experience with models of this type, and the refinements to which the adjustment of the model is carried.

8. Test procedure. After the model has demonstrated its ability to reproduce conditions similar to those which occurred in the prototype, and before the tests of various plans are undertaken, a "base test" is conducted. For this test the model is usually operated by reproducing typical, or average, annual stage and discharge hydrographs until developments within the model channel resulting from the flow conditions reproduced have become stable. Improvement plans are tested with the same hydrograph used in the base test, and the results of the latter form the basis for determining the relative effectiveness of the various plans. However, under certain conditions tests might also be conducted by reproducing a hydrograph including an unusually low- or high-water period.

Models used in study of Arkansas River problems

9. Hydraulic models of the movable-bed type have been concerned mostly with the investigation of specific problems, and the scopes of these studies generally have not been sufficiently broad to permit the development of basic information applicable to streams in general. Some of the more versatile types of movable-bed model studies have been concerned with problems in connection with the canalization of the Arkansas River. Studies completed or in progress are concerned with such problems as design and location of lock and dam structures, effects of the structures on the movement of sediment, development of channel depths and alignment, radii of curvature of bends, effectiveness of various types and arrangements of regulating structures, and navigation conditions in the approaches to the locks.

10. The models used for the study of problems in the Arkansas River have been constructed to fairly large scales to permit studies to be made in greater detail and to facilitate the study of the varied problems involved. The horizontal scales used in these models have ranged from 1:120 to 1:250 with vertical scales varying from 1:36 to 1:80; distortion of the linear scales (vertical scale divided by the horizontal scale) has varied from 1.5 to 4.17. The time scales developed for these models were such that one day of flow in the prototype was reproduced in from 5 to 10 minutes; the reproduction of an annual hydrograph would therefore require from 30.4 to 60.8 hours of operation. The test of a modification usually

requires several reproductions of the annual hydrograph, depending upon the extent of change produced by the modification, before definite indications are established. Since the time required to test a given plan is appreciable and the time available before the information is usually needed by the design engineer is limited, these studies of necessity have to be confined generally to the development of a practical solution rather than to the development of a refined optimum solution.

11. The results of studies in connection with the canalization of the Arkansas River have brought out some information which should be applicable to the regulation and improvement of natural streams in general. These studies have shown how the use of spur dikes particularly on the concave side of a bend can produce considerable disturbance to flow, reduce channel efficiency (increase roughness), and could produce shoaling upstream and downstream. It has been demonstrated that dredging of the channel bed without other changes would produce only a temporary effect, regardless of the size of the cut. The studies have emphasized the importance of the lower reach of a bendway on developments downstream. The depth and alignment of the channel over a crossing can be improved and the depth of scour and current attack along the revetted bank in a bend can be reduced by proper design of the lower reach of the bend upstream.

Effect of water temperature

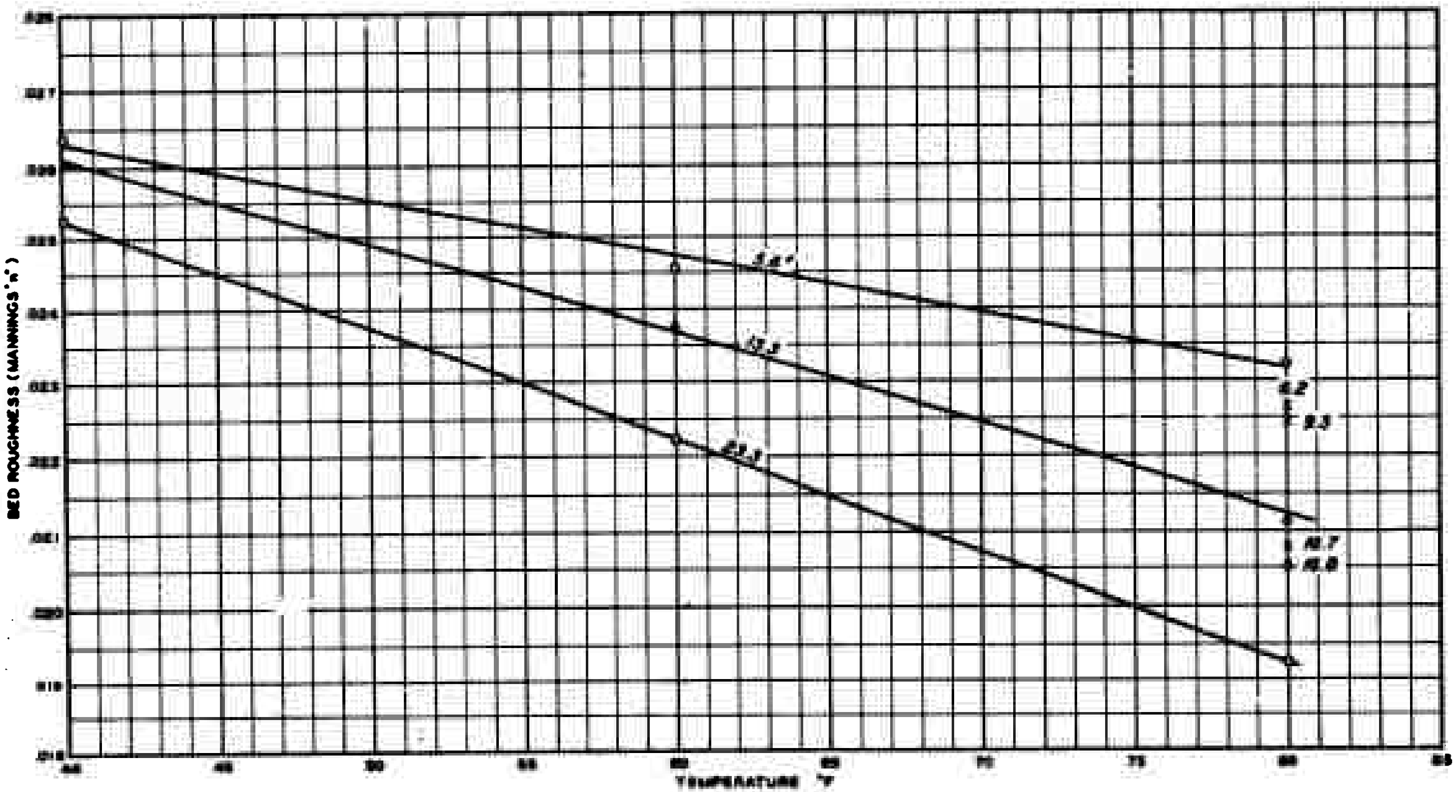
12. It has long been noted that the movement of bed material in movable-bed models can be affected by the temperature of the water. With materials of a certain grain size, riffles would form with cold water and not form with water at a higher temperature. It was also noted that the temperature at which a given material would riffle could be lowered by increasing the median grain size of the material. It has been suspected from this that the differences in the stage-discharge relation observed in alluvial streams might be caused by the difference in the temperature of the water. Research investigations are in progress to determine the effect of water temperature on the movement of bed material, bed formations, and channel efficiency.

13. This research is being conducted in a tilting flume equipped with a water circulating system, water temperature control, automatic bed-material feeder, and a device which automatically records the weight of the

material extruded. Tests are being conducted with a constant discharge, rate of bed feed, and tailwater elevation for each of three water temperatures (40, 60, and 80 deg). The flume is operated until the rate of material extruded and the water-surface slope have become stabilized for each test condition. The first series of tests has been conducted with a fairly uniform and having a median grain diameter of about 0.2 mm. A considerable amount of data was obtained during these tests, but analysis of the results has not progressed sufficiently to provide definite conclusions. Preliminary plots of some of the data obtained show an increase in the energy gradient with a decrease in the water temperature for the same rate of bed movement (fig. 1). This could be an indication that the carrying capacity of the water decreases with temperature or that the formations (riffles) along the bed are changed so as to increase the roughness of the bed as the temperature of the water is lowered (fig. 2).

14. The material used in the first series of tests riffled at all three of the water temperatures tested. It is planned to conduct the next series of tests with a material which does not riffle at those temperatures. A comparison of the results of the two series of tests might provide some of the answers.

Fig. 1. Energy gradient-temperature curves, fine sand



† NUMBER INDICATES DRY WEIGHT OF MATERIAL INTRODUCED IN LB PER HR PER FT OF WIDTH.

Fig. 2. Bed roughness-temperature curves, fine sand